Improving Subdrainage and Shoulders of Existing Pavements

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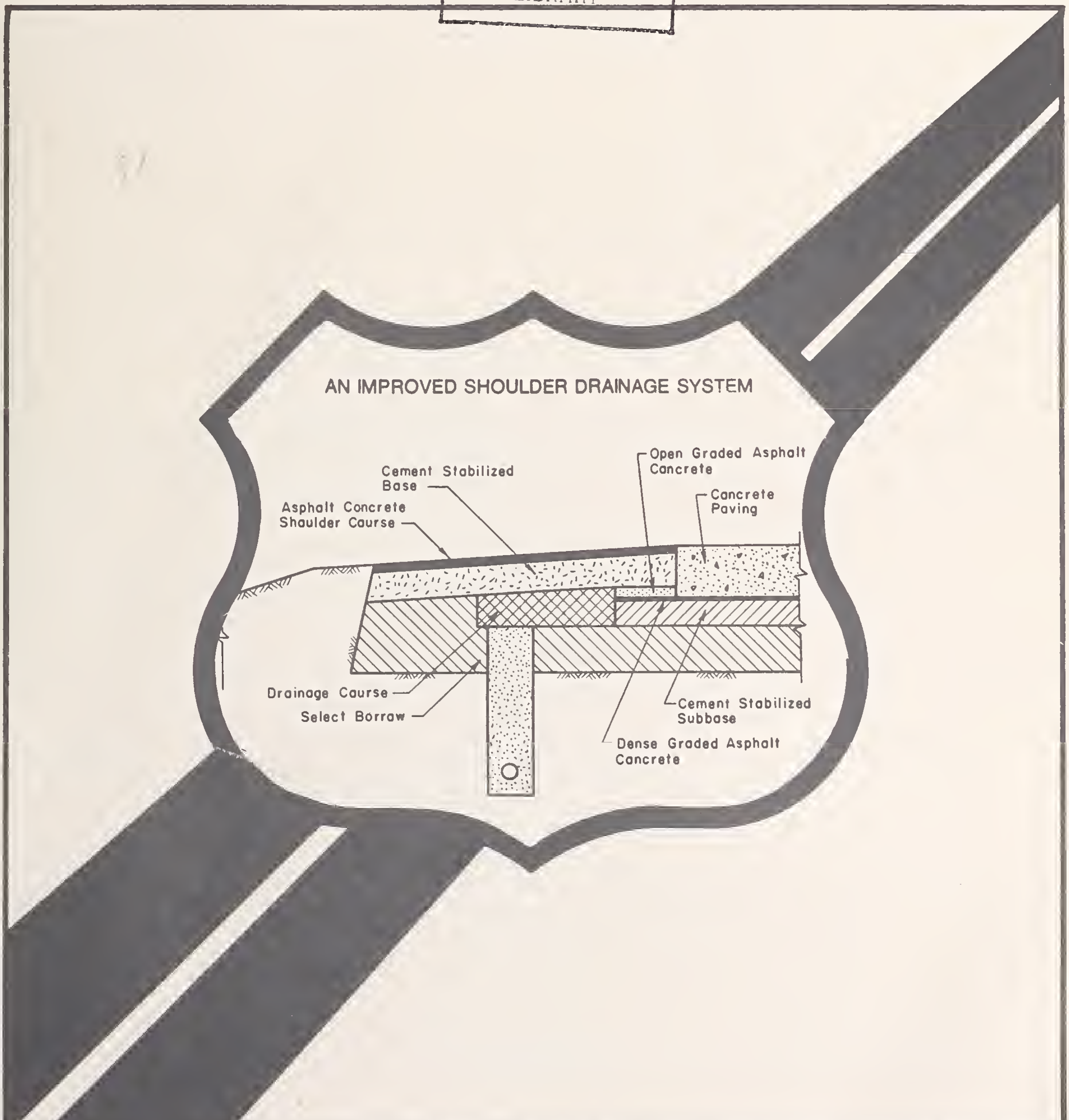

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FOREWORD

This report presents part of the results of research conducted by the University of Illinois for the Federal Highway Administration (FHWA), Office of Research, under contract DOT-FH-11-9175. This report, part of a research study on FCP Project 6D, "Structural Rehabilitation of Pavement Systems," will be of primary interest to research and development audiences concerned with subdrainage practices and shoulder designs in existing pavements.

This report supplements the following four reports related to this study. Requests for copies of these four volumes should be directed to the Pavement Division, HNR-20, Federal Highway Administration, Washington, D.C. 20590. Requests will be filled while the limited supply lasts.

FHWA/RD-81/078, "Final Report - Improving Subdrainage and Shoulders of Existing Pavements"

FHWA/RD-81/079, "A Pavement Moisture Accelerated Distress (MAD) Identification System - Volume I"

FHWA/RD-81/080, "A Pavement Moisture Accelerated Distress (MAD) Identification System - Volume II (User Manual)"

FHWA/RD-81/122, "Structural Analysis and Design of PCC Shoulders."

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

This state-of-the-art report reviews the existing literature and current practices relating to subdrainage, shoulder structures, and maintenance of existing pavement systems.

Methods for classifying the various types of pavement subdrainage are presented. Problems related to subdrainage are discussed in detail. The flow equations which describe saturated and unsaturated water movement in pavement materials and subgrades are discussed. Procedures presently being used for subdrainage design and construction have been presented.

The function, problems and design of pavement shoulders have been described in detail. Examples of shoulder designs used by several States are presented.

Maintenance practices affecting shoulder performance and moisture related distress are presented. A number of maintenance practices presently being used to decrease the effects of moisture on pavement distress are thoroughly discussed.

The general findings of the literature review are summarized and future research needs are recommended.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

MULTIPLY	BY	TO OBTAIN
inches	2.54	centimeters
feet	0.3048	meters
square inches	6.4516	square centimeters
square yards	0.83612736	square meters
knots	0.5144444	meters per second
pounds	0.45359237	kilograms
kips	0.45359237	metric tons
pounds per cubic foot	16.018489	kilograms per cubic meter
pounds	4.448222	newtons
kips	4.448222	kilonewtons (kN)
pounds per square inch	6.894757	kilopascals
pounds per cubic inch	2.7144712	kilopascals per centimeters
gallons (U. S. liquid)	3.785412	cubic decimeters
Fahrenheit degrees	5/9	Celsius degrees of Kelvins*

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F-32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F-32) + 273.15$.

DESCRIPTION OF SYMBOLS

Symbols	Description	Typical Units
C	Correction factor	
C	Pavement slope factor	
c	General mass heat capacity designation	Btu/lb-F
D	Depth of frost penetration	ft
D	Drainage width	ft
$D(\theta)$ or D_θ	Isothermal moisture diffusivity	cm^2/sec
D_T	Thermal moisture diffusivity	$cm^2/sec/c$
F	Air freezing index	°F
g	Acceleration of gravity	ft/sec^2
H	Hydraulic gradient	ft/ft
H	Total water potential	ft, cm
H	Thickness of drainage layer	ft, in.
h	Positive or negative hydraulic potential	ft, cm
ΣI	Summation of inflows	ft^3/hr, cm^3/sec
i	Hydraulic gradient	ft/ft
K	Saturated hydraulic conductivity	ft/hr, cm/sec
$K(\theta)$	Unsaturated hydraulic conductivity	ft/hr, cm/sec
K_i	Thermal conductivity of frozen material	Btu/ft-hr-F
k	Coefficient of permeability	ft/hr, cm/sec
L	Latent heat of fusion	Btu/ft^3
Le	Effective drainage path length	ft
n_d	Number of equipotential drops	
n_e	Effective porosity	cm^3/cm^3
n_f	Number of flow channels	
ΣO	Summation of outflows	ft^3/hr, cm^3/sec

DESCRIPTION OF SYMBOLS (continued)

Symbols	Description	Typical Units
Q	Flow volume	Ft^3/hr, cm^3/sec
q	Discharge veolocity or flux	ft/hr, cm/sec
T	Temperature	°F, °C
T_{50}	Dimensionless factor	
t	Time	hr, min, sec
t_{50}	Time for 50 percent drainage	hr, min,sec
v_s	Temperature difference between surface and freezing point	°F, °C
X, Y	Horizontal coordinates	
z	Vertical coordinate	
α	Thermal diffusivity	ft^2/hr, cm^2/sec
θ	Volumetric water content	cm^3/cm^3
θ	Time	hr, min, sec
λ	Dimensionless Parameter	
μ	Viscosity	Kg/m · sec
ρ	Fluid density	pcf, g/cm^3
σ	Differential operator	
AB	Aggregate base	
AC	Asphalt concrete	
ASB	Aggregate subbase	
ATB	Asphalt treated base	
COMP	Composite Pavement	
CRCP	Continuously reinforced concrete pavement	
JPCP	Jointed plan concrete pavement	
JRCP	Jointed reinforced concrete pavement	
CTB	Cement treated base	

Symbols	Description	Typical Units
FLEX	Flexible pavement	
LTB	Lime treated base	
LTS	Lime treated subgrade	
PCC	Portland cement concrete	
SA	Sand asphalt	
ST	Surface treatment	

Chapter 1

INTRODUCTION

When considering the improvement of pavement subdrainage and shoulders, the engineer is chiefly concerned with the factors of climate, loading conditions, and material properties. Water is a fundamental variable in most problems associated with pavement construction, design, behavior, and performance. Moisture is usually very significant in affecting pavement systems since the structural section as well as the subgrade are often susceptible to large moisture content variations and strongly influenced by surrounding climatic conditions.

The engineering problems associated with the behavior of pavement materials responsive to moisture content changes indicate that further study is needed with reference to the construction, design, maintenance, and rehabilitation of pavement systems which are subjected to moisture changes. The seepage of water into the structural pavement section and subgrade is one of the major contributing factors to serviceability loss in pavement systems. Frost action which requires a source of water can cause considerable volume change and damage to pavements located in cold climates. Shrinkage and swelling problems in many fine-grained subgrade soils are also attributed to moisture changes.

Water can have a major influence on the performance of pavement surfaces. Slab deterioration in Portland cement concrete pavements are often accelerated by water through "D" cracking in several areas of the U.S. Numerous joint and slab distresses are related to pumping of rigid pavements. Pumping has been observed to have a detrimental effect on the shoulder performance as well. Also, many of the distresses observed in asphalt concrete

pavements are caused or accelerated by water. Water related deterioration is a major contributing factor to the maintenance costs of Portland cement concrete pavements and asphalt concrete pavements.

Pavements deteriorate or exhibit distress over time and with traffic in a manner as illustrated in Curve A of Figure 1.1. The condition index is a relative index of the amount, type, and severity of distress. A high condition index indicates a pavement in excellent condition with no distress. For a given climate, soils, and traffic, the rate of deterioration may vary widely depending upon the pavement structure and materials as illustrated by Curves A, B and C. If the pavement structure, materials and foundation is held constant, the rate of deterioration could vary depending on climate as illustrated in Figure 1.2. Climate (or temperature, moisture, freeze-thaw, corrosion, etc.) could affect the rate of pavement deterioration through structure, durability, and/or other ways.

The concern of this research is to determine whether or not moisture is accelerating the deterioration of the pavement and if so, at what rate? For example, considering a given pavement structure, materials, soils, traffic, and temperature condition, Figure 1.3 illustrates the possible increase in the rate of deterioration due to moisture. The rate of occurrence of most distress types is nearly always a result of complex interactions of several factors including load, moisture, temperature, freeze-thaw, corrosion, etc. Thus, it cannot be concluded that "moisture alone caused this distress" in most cases, since the distress is really caused and propagates by several factors. Therefore, to determine the effect of moisture on pavement performance, its effect on accelerating the "rate of occurrence" of distress must be established. This, then, will give the true impact of moisture on pavement performance. When a specific pavement is examined to determine whether

moisture is a problem, the question that should be asked is: "has moisture accelerated the development of this distress, and if so what is the rate of acceleration?" The rate of moisture acceleration of distress can be determined by a pavement condition index that is based upon pavement distress.

It is evident from field examinations of various pavement and shoulder distresses that improved methods of establishing criteria for identifying pavements which require subdrainage are needed. It is important that guidelines for making improvements in subdrainage and shoulders be developed for various types of existing pavement systems as well as for new pavements. Furthermore it is important to determine the optimum time for making, improvements in the drainage and shoulders of existing pavement systems.

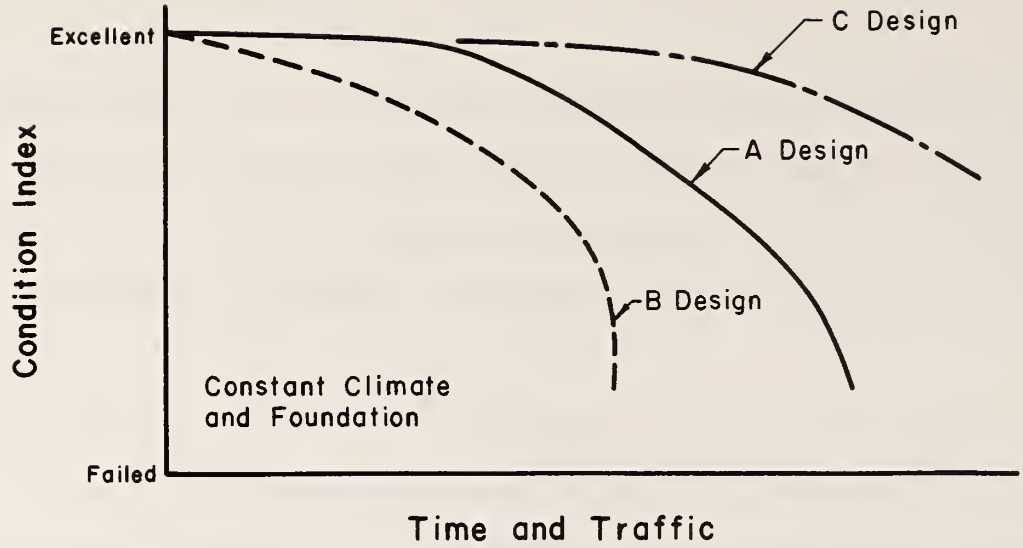


Figure 1.1. Variation in Rate of Deterioration Due to Different Structural Designs A, B, and C (in Same Climate and Traffic).

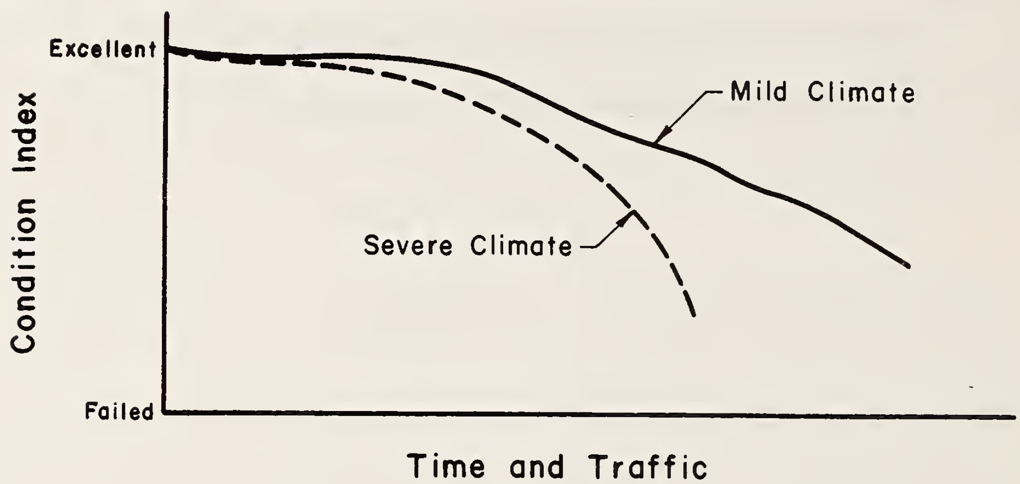


Figure 1.2. Variation in Rate of Deterioration Due to Differing Climates (Same Structure, Materials, and Traffic).

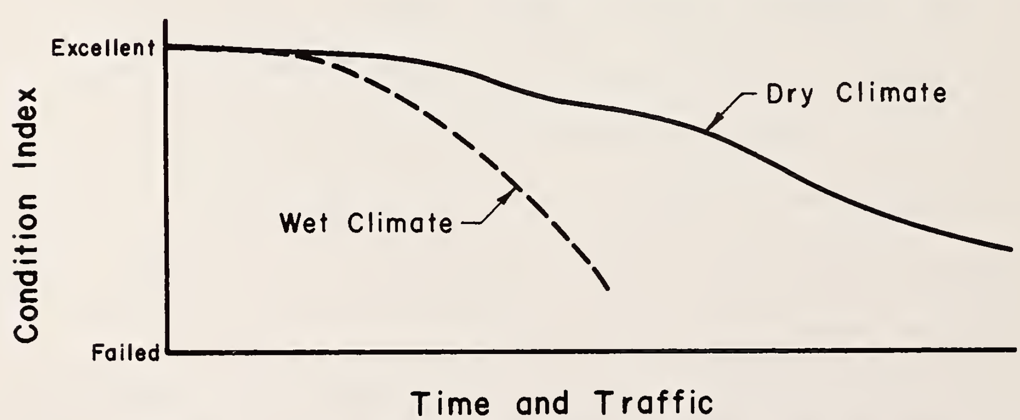


Figure 1.3. Variation in Rate of Deterioration Due to Differing Moisture Conditions (Same Structure, Materials, Temperature and Traffic).

Chapter 2

STATE-OF-THE-ART OBJECTIVES

The general objectives of this report are to review the existing literature and current practices relating to subdrainage, shoulder structures, and maintenance on existing pavement systems.

The specific areas covered by this report are as follows:

1. Classify the types of pavement subdrainage and describe the problems, flow equations, design, and construction of subdrainage systems.
2. Describe the function, problems, and design of pavement shoulders.
3. Summarizes various maintenance practices that have a significant effect on minimizing the occurrence and propagation of moisture related distress.

Chapter 3

SUBSURFACE DRAINAGE

3.1 HISTORICAL REVIEW

The problem of water in pavements has been of concern to engineers for a considerable period of time. McAdam (1) in 1823 reported on the importance of keeping the pavement subgrade dry in order to carry heavy loads without distress. He discussed the importance of maintaining an impermeable surface over the subgrade in order to keep water out of the subgrade.

More recently Cedergren and O'Brien (2) have listed 255 abstracts of pertinent literature related to the subject of subdrainage. They indicated that 26 of the abstracts were from selected references which they considered important since they presented significant, original, basic, fundamental, historical information and data regarding the subject of subdrainage.

Recent works concerning subdrainage and water in pavement systems have been published by Slaughter (3), Organization for Economic Cooperation and Development (OECD) (4), Cedergren (5) Federal Highway Administration (FHWA) (6), Barenberg and Tayabji (7), Majidzadeh (8), Dempsey (9), Dempsey and Elzeftawy (10), Ring (11), Woodstrom (12), and Barksdale and Hicks (13).

Slaughter (3) evaluated the methods currently used by highway agencies in the United States to remove subsurface water so that he could make subdrainage recommendations to the State Highway Department of Georgia. He also listed recommendations for further research needs in the area of subdrainage.

The OECD (4) has described numerous methods for predicting and controlling water in pavement systems. The OECD (4) was concerned with both saturated and unsaturated moisture movement in pavement systems. Cedergren (5) has described procedures for estimating water inflows and outflows in pavement

systems and has established criteria for designing subsurface drains. The FHWA (6) has outlined procedures for designing subsurface drainage systems for the highway structural section.

The work of Barenberg and Tayabji (7) was concerned with the evaluation of typical pavement drainage systems using open graded bituminous aggregate mixture drainage layers. Six pavement test sections incorporating different design concepts were tested in the University of Illinois Test Track. Dynamic loading was applied to the test sections and water was passed through the open graded bituminous aggregate mixture drainage layers to simulate surface and lateral infiltration. The results of the test indicated that open graded bituminous aggregate mixtures possess a very high permeability but that care must be taken to prevent the migration of subgrade fines into the drainage layer.

Majidzadeh (8) indicated that moisture and drainage related pavement problems are quite significant in Ohio. Dempsey (9) and Dempsey and Elzeftawy (10) have done considerable work in modeling moisture conditions in pavement systems. They have considered moisture movement in both the saturated and unsaturated states. Ring (11), Woodstrom (12), and Barksdale and Hicks (13) have studied the problem of water infiltration through the cracks and joints of concrete pavement systems. They have indicated that the performance life of many concrete pavements could be extended by improved drainage of the structural section.

Moulton (14) is presently working on a detailed highway subdrainage manual for the FHWA. This manual will consider subdrainage analysis and design, pavement drainage evaluation, groundwater control techniques, and construction and maintenance procedures.

Although water is a problem in pavements in itself, it can become a more severe problem in areas where frost action or freeze-thaw cycles occur.

Johnson (15) has referenced more than 800 publications which deal either entirely or partly with problems of soil freezing or frost damage. Lovell and Herrin (16) have also summarized much of the earlier knowledge on soil freezing and related problems. Researchers such as Dempsey (9, 17), Christison (18), and Berg (19) have referenced many of the more recent publications concerning temperature effects on pavement systems. Johnson, Berg, Carey, and Kaplan (20) have discussed procedures for pavement design, construction, and maintenance in seasonal frost areas. They indicated that the two most commonly used procedures for repair of frost-damaged roads are removal of the frost-susceptible soil and replacement with nonfrost-susceptible material and/or installation of a subsurface drainage network.

It is evident, based on previous research, that subdrainage is an important consideration in pavement design and performance.

3.2 CLASSIFICATION OF PAVEMENT SUBDRAINAGE

Low and Lovell (21) have indicated that moisture in pavement systems can come from several sources (Figure 3.1). They have generalized the concept of the source of pavement moisture as:

1. Moisture may permeate the sides, particularly where coarse-grained layers are present or where surface drainage facilities within the vicinity are inadequate.

2. The water table may rise (this can be expected in the winter and spring seasons).

3. Surface water may enter joints and cracks in the pavement, penetrate at the edges of the surfacing, or percolate through the surfacing and shoulders.

4. Water may move vertically in capillaries or interconnected water films.

5. Moisture may move in vapor form, depending upon adequate temperature gradients and air void space.

It can be concluded that one or more of the above mechanisms will influence moisture changes in pavement systems.

Moulton (14) has discussed the importance of classifying highway subdrainage. He indicated that subsurface drainage could be classified based on the following:

1. The source of subsurface water that is to be controlled by the drain.
2. The function that the drain is to perform.
3. The location and geometry of the drain.

Moulton (14) stated that the subdrainage classifications have to be put into perspective and that the associated technology be understood in order to avoid confusion when discussing subdrainage design. He indicated that a groundwater control system refers to subsurface drainage specifically designed to remove and/or control the flow of groundwater. Similarly, he described an infiltration control system as one designed to remove water that seeps into the pavement structural section. It is possible that the subdrainage may be required to control both groundwater and infiltration water.

Moulton (14) specified that, in terms of function, a subsurface drainage system would accomplish the following:

1. Intercept or cut off the seepage above an impervious boundary.
2. Draw down or lower the water table.
3. Collect the flow from other drainage systems.

Moulton (14) also indicated that a drainage system may serve one or more functions.

Moulton (14) listed drainage systems based on geometry as follows:

1. Longitudinal drains.
2. Transverse drains.
3. Drainage blankets.
4. Well systems.

He discussed each of these systems in detail and indicated that this is the most common method of subsurface drainage classification in pavement systems. He noted that these types of subdrainage could be designed to control both groundwater and infiltration as well as satisfy any of the functional requirements.

It is proposed to classify the subdrainage in this report according to geometry as outlined by Moulton (14). A brief definition of subdrainage classified by geometry is given below.

1. Longitudinal Drains: A longitudinal drain is located essentially parallel to the roadway centerline both in horizontal and vertical alignment. It may involve a trench of substantial depth, a collector pipe, and a protective filter. Figure 3.2 shows a horizontal drainage system.

2. Transverse Drains: Subsurface drains that run laterally beneath the roadway or are drilled into the cut slopes are classified as transverse drains. These drains are usually located at right angles to the roadway centerline, although in some cases they may be skewed. Figure 3.3 shows a transverse drainage system.

3. Drainage Blanket: The term drainage blanket is applied to a very permeable layer whose width and length are large relative to its thickness. The horizontal drainage blanket can be used beneath or as an integral part of the pavement structure to remove water from infiltration or to remove groundwater from gravity or artesian sources. A drainage blanket is shown in Fig. 3.4.

4. Well Systems: Systems of vertical wells are sometimes used to control the flow of groundwater and relieve porewater pressures in potentially troublesome highway slopes. In this application they may be pumped for temporary lowering of the water table during construction or left to overflow for the relief of artesian pressures. Sand filled vertical wells can also be used to accelerate drainage of soft compressible foundation materials which are undergoing consolidation (Figure 3.5).

Moulton (14) has stated that during highway construction and maintenance operations several different types of subsurface drainage may be required. He indicated that considerable care is required when designing the more elaborate and complex subdrainage systems.

3.3 PROBLEMS CAUSED BY SUBSURFACE WATER

3.3.1 GENERAL

Most studies of moisture conditions in pavement systems indicate that moisture content varies with season, climatic conditions, geologic location, and type of pavement system. Based on a study of 13 United States airfields located in nonfrost-susceptible areas, Redus (22) indicated that subgrade moisture conditions may go up or down following construction and appear to stabilize after about 2 yrs, subsequently displaying small fluctuations (increases and decreases). Low and Lovell (21) concluded from their study of current literature that the moisture content shows continuous, if small, variation with the seasons. In a long-term study of moisture conditions beneath rigid pavements in Missouri, Guinnee and Thomas (23) found that moisture variations in the top levels of the subbase and subgrade were greater than those noted at deeper levels. Chu and Humphries (24) found

that moisture contents in pavement systems in South Carolina varied with season, soil type, and location in the pavement system (Figure 3.6). They also found that the subgrade moisture content was influenced by the depth of the groundwater table (Figure 3.7). Aitchison and Richards (25), based on results from their study of moisture conditions in pavement subgrades throughout Australia, stated that moisture stability beneath the greater part of the paved area was similar for every test site even though the climatic conditions at the sites were widely different. However, a field study conducted by the Corps of Engineers (26) indicated that the amount of precipitation could have considerable influence on the moisture conditions in airport subgrade and base course materials (Figure 3.8).

Marks and Haliburton (27), based on a short-term study of typical Oklahoma highways, concluded that most moisture variations occur beneath highway pavements on an annual cycle with maximum moisture contents occurring during the winter months. Hicks (28) found that for 20 stations in North Carolina the moisture contents for the bases and subgrades were highest during later winter or early spring.

Frost action effects accentuate moisture content increases in the pavement system. Road Research Laboratory studies indicate a significant increase (1 to 3 percent) in granular base moisture content due to frost action (29). Stevens, Maner, and Shelburne (30) observed that spring pavement breakup in Virginia could be related to the amount of fall precipitation and the length of the freezing period. They found that large amounts of precipitation in October, November, and December tended to saturate the subgrade and base beneath the highways studied. Straub, Dudden, and Moorhead (31) found significant moisture increases beneath snow-covered shoulders during periods of thawing as a result of gravitational flow of snowbank melt water.

Marks and Haliburton (27) indicated that cyclic moisture content variations were considerably affected by precipitation at sites where the pavements were poor (greater degrees of cracking and perviousness in the pavement surface). Moisture content changes could not be correlated with precipitation for pavement sections with high ratings (little cracking, good surface condition). The moisture content variations for the higher rated pavements were primarily attributed to temperature effects.

The results of an analysis of the influence of precipitation on soil moisture content by Moulton and Dubbe (32) showed that the amounts of precipitation occurring at various periods prior to moisture content sampling were not statistically significant at the 5 percent level in explaning the observed variations of moisture content either in the base and subbase materials or in the subgrade soils. They found that, in West Virginia, moisture content in granular base and subbase materials was more dependent upon the drainage characteristics of the materials and the site than upon precipitation.

Observations by Turner and Jumikis (33) showed that moisture content beneath a pavement is affected by the groundwater table. They found that precipitation could modify the position of the water table and subgrade moisture content and that the degree of change was sensitive to the type of precipitation. They found that more water from melting snow precipitation percolated into the ground than if the precipitation was in the form of rain.

Several investigators (22, 23, 34) have concluded that moisture contents at the pavement edges are generally higher than those at the interior locations. Guinnee and Thomas (23) stated that water enters the pavement more easily and in greater volumes at the pavement edges. Benkelman (35), in an analysis of WASHO Road Test deflection data, correlated inner and outer

wheel path deflections with degree of subgrade saturation (Figure 3.9). According to Benkelman (35), adverse moisture conditions existed at the pavement edges. Aitchison and Richards (25) also noted the greater moisture content fluctuation at the pavement edges for Australian pavements.

Moisture content, dry density, CBR, and plate bearing data were obtained for the flexible pavements in Loop 1 (no traffic) and Loops 3, 4, 5, and 6 at the AASHO Road Test. The data presented in Figures 3.10, 3.11, and 3.12 indicate moisture content increases and CBR decreases in the base and subbase layers (36). In general, the moisture content of the subgrade (Figure 3.13) was quite stable but some decrease in CBR was noted in the spring. It was observed that the increase in CBR strength, spring to summer, of the embankment soil could not be satisfactorily explained by differences in moisture content or density. However, recent resilient modulus studies by Robnett and Thompson (37) indicate that a 1 or 2 percent moisture content change can have considerable influence on the strength of the AASHO Road Test subgrade soil. Figure 3.14 shows the effect of compaction moisture content variation on the resilient modulus of the AASHO Road Test A-6 subgrade soil and several other soils.

3.3.2 DISTRESS MANIFESTATIONS

The types of pavement distresses caused by water and temperature are quite numerous. In a report prepared for the Department of the Army Construction Engineering Research Laboratory, Barenberg, Bartholomew, and Herrin (38) identified many of the common types of pavement distresses. Some of the distresses related to temperature and moisture either singularly or in combination are:

1. Flexible pavement systems
    a. Potholes
    b. Loss of aggregates
    c. Raveling
    d. Weathering
    e. Alligator cracking
    f. Reflective cracking
    g. Shrinkage cracking
    h. Shoving
    i. Frost heave
2. Rigid pavement systems
    a. Faulting
    b. Joint failure
    c. Pumping
    d. Corner cracking
    e. Diagonal cracking
    f. Transverse cracking
    g. Longitudinal cracking
    h. Shrinkage cracking
    i. Blowup or buckling
    j. Curling
    k. D-cracking
    l. Surface spalling
    m. Steel corrosion

Cedergren (39) has observed similar types of distresses in taxiways and runways of numerous airfields in the United States. Johnson, et al. (20),

Darter and Barenberg (40), and Shahin, Darter, and Kohn (41) have also observed many of the distress manifestations in flexible and rigid pavement systems.

Dempsey (9), Thompson (42), and Thompson and Dempsey (43) have cited several examples of how temperature and moisture influence the behavior and performance of pavement systems.

Extensive studies of flexible pavement sections by the Canadian Good Roads Association (44) have shown that the average maximum spring rebound deflection value is equal to 1.63 times the average fall rebound value. In some cases, the spring values were 5.2 times larger than the fall values. Figure 3.15 shows the deflection history of a pavement in Minnesota (45). The pavement section was 3 in. of asphaltic concrete surface, 3 in. of crushed rock base, and 9 in. of sand-gravel subbase. From Figure 3.15 it is quite apparent that temperature and moisture have considerable influence on pavement deflection. The approximate Benkelman Beam rebound deflection for an 18-kip axle load can be calculated by multiplying the Dynaflect values in Figure 3.15 by 20.

Extensive plate-loading field test studies for several states in freezing zones have shown that spring bearing values are substantially less than fall values (46, 47, 48, 49). Although the data displayed substantial variability, for many of the flexible pavement sections the spring values were as low as approximately 40 percent of the fall values. The load-carrying capacity (30-in. plate, 0.1-in. deformation) of concrete runways studied by Linnel and Halel (50) was reduced during the spring frost-melting period to approximately 70 percent of the normal load-carrying capacity.

Ring (51) has indicated that spring subgrade bearing strengths in the northern States range from 30 percent to 100 percent of the fall values (Figure 3.16).

Benkelman (52) reported that a great deal more structural deterioration took place during the spring months than during the summer months in the AASHO Road Test flexible pavements. A seasonal weighting factor was developed at the AASHO Road Test to account for the relative effects of axle loads applied at different times of the year (36). The factor, applied only to the flexible pavements, ranged from 0 (no damaging effect) to 4.84 (very high damaging effect). The factors were low for summer months and were maximum during the spring when the detrimental effects of frost action and moisture were greatest.

Kingham (53) has indicated that Benkelman Beam deflections in a full-depth asphalt pavement could change as much as 0.001 in. for a temperature change of 1°F in the asphalt concrete.

Cumberledge, Cominsky, and Bhajandas (54) related pavement surface deflection measured with the Road Rater to the moisture content in the base and subgrade and to precipitation in Pennsylvania (Figure 3.17). They found that the deflection generally increased as the moisture content increased, except during the winter months when the deflection was constant due to the frozen condition of the pavement base and subgrade.

Pumping, a major problem related to jointed rigid pavement performance, is associated with deflection and excessive moisture contents in the support materials beneath the slab. All failures in the rigid pavement sections at the AASHO Road Test were preceded by pumping of material from beneath the concrete slab (55). Both Yoder (56) and Cedergren (39) noted damage to airport runways and taxiways caused by pumping. These observations were

especially prevalent where pavement overload and channelized traffic had occurred.

Woodstrom (12) has found that faulting at the transverse joints is a normal manifestation of distress of unreinforced concrete pavements without load transfer in California. He indicated that for faulting to occur the following conditions must exist:

1. The pavement slab must have a slight curl with the individual slab ends raised slightly off the underlying stabilized layer (thermal gradients and differential drying within the slab create this condition).
2. Free water must be present.
3. Heavy loads must cross the transverse joints first depressing the approach side of the joint, then allowing a sudden rebound, while instantaneously impacting the leave side of the joint, causing a violent pumping action of free water.
4. Pumpable fines must be present (untreated shoulder base material, the surface of the stabilized base, and foreign material entering the joints can be classified as pumpable fines).

Woodstrom (12) indicated that faulting of 1/4 in. or more adversely affected the riding quality of the pavement system.

In considering significant mileage of portland cement concrete pavements with adjoining asphalt concrete shoulders, Barksdale and Hicks (13) have indicated that deterioration of the shoulder in the vicinity of the horizontal pavement-shoulder joint is considerably more severe when a significant quantity of water is present beneath the pavement and shoulder structure. They indicated that the important factors affecting the severity of the

water-associated problems include the amount and distribution of rainfall, temperature, expansive clay subgrades, and the strength and type of structural shoulder section. Barksdale and Hicks (13) have stated that it is possible for as much as 70 to 97 percent of the rainfall to enter the open joints of concrete pavements.

Field infiltration measurements indicated that even under prolonged wet conditions, a pavement resting on a moderately impermeable subgrade could have an average infiltration capacity of about 30 percent of the rain falling on the pavement. Barksdale and Hicks (13) cautioned that the pavement surface condition may not be indicative of infiltration rate, but that conditions existing beneath the pavement may control the amount of water entering the structural section.

Ring (11) has stated that water infiltrating through cracks and joints and becoming trapped within the structural section of rigid pavement systems appears to be a major factor in the performance of many concrete pavements. There is often a loss of subgrade support and pavement faulting caused by redistribution of subbase material.

3.3.3 MECHANISMS OF INFLUENCE

Thompson (42) indicated that the mechanisms whereby climate may influence pavement behavior and performance can be stated as:

1. Effect on the engineering properties of component materials.
2. Disintegration of materials caused by temperature and moisture changes (durability failure).
3. Temperature- and moisture-induced volume changes in component materials.

<u>Effect on the Engineering Properties:</u> The general response of a pavement system to traffic loading is controlled by the thickness of the various structural layers and the significant engineering properties of the paving materials and the subgrade soil. It is apparent that if engineering property changes are affected by water and temperature, pavement response will likewise be influenced.

Temperature and water content both significantly affect the engineering properties of soils. A recent International Conference Proceedings on soils (57) is devoted solely to the topic, "Effects of Temperature and Heat on Engineering Behavior of Soil." Strength and elasticity are two properties of particular importance to pavement soil problems. Lower strengths are generally associated with higher temperatures which cause changes in the soil and water properties. (57).

Bergan and Fredlung (58) found that freeze-thaw cycles could influence the resilient modulus of a clay soil (Figure 3.18). Similar results were also acknowledged by Robnett and Thompson (37) in studies of the resilient behavior of fine-grained soil. Figure 3.19 shows the results of freeze-thaw cycles on the resilient behavior of a clay soil. From Figure 3.19 it is noted that additional loading and curing periods had minimal strengthening effect on the resilient behavior of the soil after freeze-thaw cycles.

Moisture content has a pronounced effect on the strength and deformation properties of soils. CBR-moisture content-density relations for the AASHO Road Test embankment soil (Figure 3.20) illustrate the typical effect (59). Extensive studies by Croney, Coleman, and Black (60) in England have emphasized the development of relations between soil suction and soil stability (Figures 3.21 and 3.22). Sauer and Monismith (61) have studied the repeated-load behavior of a glacial till at different suctions (moisture contents). Resilient moduli were substantially affected by soil suction (Figure 3.23).

Barenberg and Tayabji (7) found that subgrade saturation was a major cause of pavement failure in a test track study of drainage of an open-graded base material.

Water content, rather than temperature, is the major climatic factor influencing the strength and deformation properties of granular materials. Haynes and Yoder (62) found that the degree of saturation had a substantial effect on the repeated-load deformation properties of the AASHO Road Test crushed stone and gravel materials (Figure 3.24).

A recent study by Thompson (63) emphasized the behavior of granular materials in pavement systems. As a result of the study, it was concluded that granular materials at high levels of saturation become unstable under repeated loading. The importance of the subgrade soil condition (moisture content, strength) at the base course-subgrade interface was emphasized. Model study results indicated that bonded interface conditions promoted improved repeated-load behavior in the granular layer and the pavement system (63).

Substantiating data illustrating the detrimental effect of high moisture content on the repeated-load behavior of granular materials have been presented by Snyder (64). The importance of moisture content relative to granular material stability can be inferred from the many field studies of spring breakup which is typical in freezing zones.

The strength and deformation properties of bituminous materials and bituminous mixtures are substantially influenced by temperature. The effect of temperature on the Marshall stability of the AASHO Road Test surface and binder course asphaltic concrete mixtures is illustrated in Figure 3.25 (36). Flexural stiffness is also substantially influenced by temperature as shown in Figure 3.26 (65). Van der Poel (66) has developed

a procedure for determining the stiffness of bitumen which considers the effects of temperature and time of loading.

Monismith and Deacon (67) have indicated that relatively small temperature changes can drastically alter the fracture life of an asphalt paving mixture. Pell's data (68), presented in Figure 3.27, illustrate this point.

Marek and Dempsey (69) and Dempsey and Marek (70) have shown that stress and deformation properties of flexible pavement systems are influenced by the temperature of the asphaltic concrete surface. Figure 3.28 shows typical effects of seasonal temperature changes on stress.

Durability Failure: Typical paving materials (concrete, aggregates, asphaltic concrete, and stabilized soils) are susceptible to deterioration due to the action of environmental factors (primarily temperature, moisture, and in the case of asphalt, sunlight). It is beyond the scope of this report to evaluate the durability properties of all paving materials. In some instances climate-induced deterioration progresses to the extent that the materials are almost completely disintegrated. It is possible to have a pavement failure caused primarily by climatic factors and not by wheel loading.

Barenberg, Bartholomew, and Herrin (38) have identified most of the durability failures in flexible and rigid pavement systems. Thompson and Dempsey (71) have discussed the freeze-thaw durability problems related to materials stabilized with lime, lime-flyash, and cement.

Corrosion of reinforcing steel and dowel bars is a source of durability failure in rigid pavement systems, especially in regions where deicing salts are used in large quantities (72). Consideration must be given to the durability characteristics of the component materials utilized in the pavement system. These durability characteristics are a function of the

climatic conditions, geographical location, pavement materials, and position in the pavement system.

Temperature and Moisture Induced Volume Changes: The mechanisms of frost heaving have been described by Jumikis (73), Jessberger (74), Kaplar (75), Shober (76), and Dempsey (9). In general, frost heaving is caused by the growth of ice crystals and not to a large extent by the change in volume of water when freezing. Pressure is developed in the direction of crystal growth which is usually in the direction of the temperature gradient. The total amount of frost heaving is in direct proportion to the increase in water content which influences the thicknesses of the ice crystals and ice lenses in the frozen soil.

Jessberger (74) has indicated that frost heaving results when water is pulled through the soil to build layers of lenticular masses of segregated ice which grow in thickness because a larger crystal already present will continue to grow over a newly formed crystal. The heave may be uniform or nonuniform depending on the soil characteristics and groundwater. In uniform heave the pavement moves evenly and the shape and smoothness of the surface remain substantially unchanged. In nonuniform heave the pavement surface experiences objectionable irregularities and unevenness. Figure 3.29 shows some of the mechanisms involved in the growth of ice lenses in soils.

It is generally recognized that the conditions necessary for frost heaving to occur in the subsurface pavement materials are:

1. The soil must be frost susceptible.
2. Freezing temperatures must penetrate the soil.
3. Water must be available.

Shober (76) has indicated that differential frost heaving in the vicinity of the pavement joints and cracks can cause frost tenting. He indicated that the conditions necessary for frost tenting are as follows:

1. Temperatures below freezing.
2. Salt in solution entering pavement crack.
3. Free-water supply.
4. Saturation of pavement base and subgrade.

Carpenter, Lytton, and Epps (77) found that granular base course materials under asphaltic concrete may experience volume changes as a result of freeze-thaw cycling. They indicated that the volume changes in the base can cause tensile cracks which can reflect through the asphaltic concrete to the surface. Temperature cracking is a major problem in asphaltic concrete pavements in cold climates. Johnson, et al. (20) have discussed the mechanisms causing low-temperature or frost-induced pavement cracking.

The mechanisms of volume change in swelling clays and shales have been described by Kassiff, Livneh, and Wiseman (78) and summarized in a workshop proceedings by Lamb and Hanna (79). The observed volume changes in swelling clays and shales are generally closely related to the changes in the moisture content.

Tourtelot (80) has indicated that volume changes in expansive clays and shales can occur on a megascale or a microscale (Figure 3.30). On the megascale, volume change is due to the response of the soil to overburden pressures which occur during the geologic history or stress history of the deposit. Heave occurs when overburden pressures are removed from a highly consolidated foundation material, and settlement occurs when the overburden

pressure exceeds the consolidation pressure of the foundation material. The volume process is related to the free water, absorbed water, adsorbed water, and particle arrangement of the soils in a deposit. Tourtelot (80) has indicated that microscale volume changes in clay soils can be related to the hydration and desiccation process. Figure 3.31 shows diagrammatrically the nature of hydration volume changes which are a function of the physical characteristics of clay. The volume change process as shown in Figure 3.31 can be both interparticle and intracrystalline. The process is also reversible. On drying, the soil particles are pulled together under tension, and on wetting the tension decreases and the soil expands.

The swell capacity of various clay minerals is shown in Figure 3.32. Sodium montmorillonite has the highest swelling potential.

Low (81) has discussed the volume change mechanism involved in expansive clays. He has indicated that the swell potential in clay minerals is largely a function of layer spacing.

Lytton, Boggess, and Spotts (82) indicated that pavement roughness caused by expansive clay appears to be predictable from mineralogical and pedologic properties of the clay deposit. They indicated that density, water content, and suction were important parameters to be considered in evaluating expansive soils.

The end results of most volume changes caused by swelling soils or frost action are rough pavement surfaces and cracking types of failures. The cracks are conducive to the infiltration of water into the structural pavement sections which can lead to accelerated deterioration.

3.4 EQUATIONS AND MODELS FOR DESCRIBING MOISTURE AND TEMPERATURE REGIMES IN PAVEMENTS

3.4.1 SATURATED MOISTURE FLOW

In mechanics the motion of a fluid can be described in terms of a kinematic, dynamic, and thermodynamic relationship. The first of these relationships leads to a statement of conservation of mass, the second can be derived from a momentum balance, and the third takes the form of an equation of state. For the viscous flow of fluids through a porous media, classical theory leads to the Navier-Stokes equations. However the Navier-Stokes equations do not lend themselves to solution of many practical problems so a dynamic relation derived from experimental observations known as Darcy's Law is generally used. Cedergren (5) has given a number of different forms of Darcy's law. A common form of the law is as follows:

$$q = Ki \tag{3-1}$$

In Equation 3-1, q is the discharge velocity or flux, K is the hydraulic conductivity, and i is the hydraulic gradient.

Where flow is unsteady or the soil nonuniform, the hydraulic head may not decrease linearly along the direction of flow. Where the hydraulic head gradient, flux, or conductivity is variable Darcy's law can be expressed in a generalized differential form as follows:

$$q = -K\nabla H \tag{3-2}$$

In the differential form, Equation 3-2, K is the hydraulic conductivity, H is the hydraulic gradient, and q is the flux or flow velocity. In Equation 3-2 q is a vector, K is a scalar, and H is a vector. It should be noted that the flow rate, q, often referred to as the flux, represents the discharge rate per unit cross-sectional area, and it is not the actual fluid velocity through the pores.

In terms of the x, y, and z directions, Darcy's Law is expressed as follows:

$$q_x = -K_x \frac{\partial H}{\partial x} \tag{3-3}$$

$$q_y = -K_y \frac{\partial H}{\partial y} \tag{3-4}$$

$$q_z = -K_z \frac{\partial H}{\partial z} \tag{3-5}$$

The hydraulic conductivities K_x, K_y, and K_z in Equations 3-3, 3-4, and 3-5, respectively, may or may not be equal. If they are all equal, the porous medium is isotropic. If not, the porous medium is anisotropic. Childs (83) has indicated that Equations 3-3, 3-4, and 3-5 are valid for anisotropic porous media only if x, y, and z are the principal axes of the medium with respect to hydraulic conductivity.

The hydraulic conductivity, K, is related to the permeability, k, of a porous medium by the following relation

$$K = k\rho g/u \tag{3-6}$$

In Equation 3-6, ρ and u are the density and viscosity respectively and g is the acceleration of gravity.

The hydraulic head, H, can be related to the pressure head and position head by the following equation:

$$H = h \pm z \tag{3-7}$$

In Equation 3-7, h is the pressure in the fluid and z is the position of the free water surface with reference to some datum.

Darcy's Law has the form of a flux proportional to a driving force and it is analogous to Ohm's Law for electricity and Fourier's Law of heat conduction.

Saturated flow may be described in terms of steady state drainage or nonsteady state drainage. Kirkham and Powers (84) and Kirkham, Toksoz, and Van Der Ploeg (85) have reviewed most of the theory concerning steady-state drainage. Van Schilfgaarde (86) has described the nonsteady or time-varying flow relationships. King (87) has discussed the equations for describing saturated flow through heterogeneous porous media.

The solution of unsteady flow processes in which the magnitude and possibly the direction of flux and potential gradient vary with time require the additional law of conservation of matter. The mass-conservation law expressed in the equation of continuity states that if the rate of inflow into a small volume element is greater than the rate of outflow then the volume element is storing the excess fluid. For multidimensional systems the equation of continuity is expressed in the following form:

$$\frac{\partial \theta}{\partial t} = - \nabla \cdot q \qquad (3\text{-}8)$$

The terms θ and t are the water content and time respectively. Combining the continuity equation, Equation 3-8, with the differential form of Darcy's equation, Equation 3-2, yields the general flow equation:

$$\frac{\partial \theta}{\partial t} = \nabla \cdot (K \nabla H) \qquad (3\text{-}9)$$

In a saturated soil with an incompressible matrix, $\frac{\partial \theta}{\partial t} = 0$, the hydraulic conductivity, K, is usually assumed to be constant, and Equation 3-9 can be expressed as follows:

$$K\nabla^2 H = 0 \tag{3-10}$$

For three-dimensional saturated flows in a material which is not isotropic Equation 3-10 results in the following:

$$K_x \frac{\partial^2 H}{\partial x^2} + K_y \frac{\partial^2 H}{\partial y^2} + K_z \frac{\partial^2 H}{\partial z^2} = 0 \tag{3-11}$$

In isotropic materials where $K_x = K_y = K_z$ Equation 3-11 reduces to the well-known Laplace equation:

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} + \frac{\partial^2 H}{\partial z^2} = 0 \tag{3-12}$$

For two-dimensional flow Equation 3-12 can be changed to the following form:

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial z^2} = 0 \tag{3-13}$$

Equations 3-12 and 3-13 are second-order partial differential equations of the elliptical type and they can be solved by various methods to obtain a quantitative description of water flow in porous materials.

Solutions to Equations 3-12 and 3-13 can be obtained by means of electric analogues, hydraulic models, numerical methods, and by graphical flow nets. Luthin (88) has discussed the use of electric analogues to solve drainage problems. Guitjens (89) has discussed in detail the use of hydraulic models. Numerical methods utilizing either finite element (90) or finite difference (91) techniques can be used to analyze a variety of complex subsurface flow problems. The major advantage of the numerical approach is that computers can be readily used to solve the drainage problems and complex drainage geometrics can be analyzed.

Cedergren (92) and Terzaghi and Peck (93) have described the use of flow nets to solve Equation 3-13. Graphically, Equation 3-13 can be represented by two sets of curves that intersect at right angles (Figure 3.33). The curves of one set are called flow lines whereas the curves of the other set are known as equipotential lines. At all points along an equipotential line the water would rise in a piezometric tube to a certain elevation known as the piezometric level. The water particles travel along the flow lines in a direction perpendicular to the equipotential lines. Figure 3.34 shows the use of a flow net to determine seepage rate through a subgrade. From the flow net the total seepage quantity can be estimated from the following equation:

$$Q = KH \frac{n_f}{n_d} \tag{3-14}$$

In Equation 3-14, Q is the flow volume per unit time, K is the hydraulic conductivity, H is the hydraulic head causing flow and the ratio n_f/n_d is the shape factor of the flow net where n_f is the number of flow channels and n_d is the number of equipotential drops.

Both Cedergren (5) and Moulton (14) have demonstrated the use of flow nets to solve a broad range of seepage problems in pavement systems.

3.4.2 UNSATURATED MOISTURE FLOW

Although subsurface drainage design is generally concerned with saturated flow in porous media, considerable water can also move by the processes of unsaturated flow. It is important to develop an understanding of the full range of mechanisms by which water can move through a material.

Darcy's Law, though originally conceived for saturated flow only, was extended by Richards (94) to unsaturated flow, with the provision that the conductivity is a function of the matrix suction head. The unsaturated flow equation is expressed as follows:

$$q = -K(\theta)\nabla H \tag{3-15}$$

In Equation 3-15, $K(\theta)$ is a function of unsaturated moisture content and H is the hydraulic head as described by Equation 3-7 except that h is a tension or suction heat instead of a positive pressure head.

Equation 3-15 has been found to be valid for a range of flow systems including unsaturated flow (steady and nonsteady) by Childs and Collis-George (95) and nonsteady flow by Rogers and Klute (96).

When Darcy's Law is applied to unsaturated flow situations, the values of the hydraulic conductivity are found to be highly variable with moisture content. Hydraulic conductivity is largest when the soil is water-saturated, decreasing as the moisture content decreases. Typically, hydraulic conductivity changes by factors of thousands in the range of moisture contents encountered in the field. Elzeftawy and Dempsey (97, 98) have presented hydraulic conductivity values of typical soils and materials in pavement systems. Bouma (99) has developed hydraulic conductivity-matric potential curves for some major soil horizons.

The laws necessary for consideration of transient flow systems are the extension of Darcy's Law to unsteady flow systems and the principle of the conservation of matter. By combining Equation 3-15 with the continuity equation, Equation 3-8, the following general transient flow equation is obtained:

$$\frac{\partial\theta}{\partial t} = \nabla \cdot [K(\theta)\nabla H] \tag{3-16}$$

Since the total head is equal to the sum of the pressure head (in this case a suction head) and gravitational head (Equation 3-7), Equation 3-16 can be written as follows for the one-dimensional condition:

$$\frac{\partial\theta}{\partial t} = \frac{\partial}{\partial z} \left[K(\theta) \frac{\partial h}{\partial z}\right] + \frac{\partial K(\theta)}{\partial z} \qquad (3\text{-}17)$$

$$\frac{\partial\theta}{\partial t} = \frac{\partial}{\partial z} \left[D(\theta) \frac{\partial\theta}{\partial z}\right] + \frac{\partial K(\theta)}{\partial z} \qquad (3\text{-}18)$$

In Equation 3-18, $D(\theta)$ is called the soil-water diffusivity and it is equal to $K(\theta)\partial h/\partial\theta$. The relationship $\partial h/\partial\theta$ is obtained from the slope of the moisture characteristic curve (Figure 3.35).

In terms of three-dimensional cartesian coordinates the transient unsaturated flow equation as given in Equation 3-16 can be stated as follows:

$$\frac{\partial}{\partial x} \left[K(\theta) \frac{\partial H}{\partial x}\right] + \frac{\partial}{\partial y} \left[K(\theta) \frac{\partial H}{\partial y}\right] + \frac{\partial}{\partial z} \left[K(\theta) \frac{\partial H}{\partial z}\right] = \frac{\partial\theta}{\partial t} \qquad (3\text{-}19)$$

For the steady-state condition, $\frac{\partial\theta}{\partial t} = 0$, and Equation 3-19 takes the following form:

$$\frac{\partial}{\partial x} \left[K(\theta) \frac{\partial H}{\partial x}\right] + \frac{\partial}{\partial y} \left[K(\theta) \frac{\partial H}{\partial y}\right] + \frac{\partial}{\partial z} \left[K(\theta) \frac{\partial H}{\partial z}\right] = 0 \qquad (3\text{-}20)$$

A more detailed discussion of unsaturated moisture flow can be found in previous work by Dempsey and Elzeftawy (10, 100) and Dempsey (9).

In general, the analysis of transient flow includes the extension of the equilibrium relation between soil-water content and matrix potential to flow situations, Darcy's Law, and the law of conservation of matter.

Mathematical models based on theoretical solution of the unsaturated flow equations using numerical methods have been developed by several investigators (10, 100, 101, 102, 103, 104, 105, 106). A rational method based on the thermodynamic theory of equilibrium distributions of water in porous media has been developed by researchers of the British Road Research Laboratory (107, 108, 109, 110). Dempsey (9) and Thompson and Dempsey (43) have discussed this method of moisture prediction.

Numerous procedures based on empirical methods have also been developed for predicting unsaturated moisture content. A procedure based on the Thornthwaite (111) moisture index is commonly used.

A thorough discussion of the theoretical and empirical methods for predicting unsaturated moisture content in pavement system can be found in previous work by Dempsey (9).

3.4.3 TEMPERATURE EQUATIONS AND MODELS

Since frost action is often associated with subsurface drainage it is important to discuss the basis equations of models for heat transfer in pavement systems.

Methods for predicting temperatures in pavement systems have progressed substantially since an awareness in this area was first manifested in the early works of Eno (112) and Sourwine (113). It has become increasingly evident that a comprehensive study of pavement temperature problems is dependent upon the ability to accurately predict the relationship between the climatic, hydrologic, and geographic conditions and the intrinsic pavement conditions.

Most of the methods for predicting temperatures in pavement systems have been developed for the purpose of evaluating frost action or determining the depth of frost penetration. Measured depths of frost penetration beneath pavements in service were initially used as a general durability guide. Later more sophisticated analytical methods were developed to determine the depth of frost penetration as well as temperature distribution and variation. Many of the recent methods developed for predicting pavement temperatures consider the thermodynamic principles of heat transfer and include radiation, convection, and conduction. These methods generally utilize the Fourier diffusion equation for determining conductive heat transfer in a pavement system. The Fourier equation in the general one-dimensional form is as follows:

$$\frac{\partial T}{\partial \theta} = \alpha \frac{\partial^2 T}{\partial x^2} \qquad (3\text{-}21)$$

In Equation 3-21, T is the temperature of the pavement at some specific depth x, and time θ. The thermal diffusivity, α, is related to the thermal conductivity and heat capacity of the pavement materials. Dempsey (17), Berg (19), Aldrich (114) and Moulton and Dubbe (32), have discussed the fundamental principles involved in the development of Equation 3-21.

Over the years a number of methods have been developed for predicting the depth of frost penetration and the temperature regime in the pavement systems. These methods can be placed in two general categories as follows:

1. The empirical methods, which utilize data from actual measurements in the field.
2. The analytical methods, which have been developed mathematically from heat transfer theory with or without simplifying assumptions.

Empirical correlations between either the air freezing index or the surface freezing index and the depth of frost penetration have been developed for pavement design purposes by the Corps of Engineers (115), Brown (116), Culley (117), and Haley (118).

Carslaw and Jaeger (119) have presented numerous solutions to one-, two-, and three-dimensional heat transfer problems. Generally, they assume that the materials used in their solutions are homogeneous and isotropic. Lachenbruck (120) developed a method for estimating the three-dimensional thermal regime in a homogeneous isotropic soil beneath a heated structure. The techniques developed by Carslaw and Jaeger (119) and Lachenbruck (120) do not consider phase changes in the soil moisture. For this reason the differences between computed and measured depths of frost penetration are found to increase rapidly as moisture content in the soil increases.

The most common methods for predicting the depth of frost penetration have been by formulas and charts based on solutions to the Fourier diffusion equation (Equation 3-21) and modified by assumptions and the use of observed data. Of the formulas which have been developed, the Stefan equation and modified Berggren equation have been the most common methods for predicting frost penetration.

The Stefan equation is the simplest of the frost penetration equations. According to Jumikis (73), the solution was first published by Stefan in 1890 as a simple formula for determining the rate of ice development in still water. In its application to soils, the Stefan equation is based on the hypothesis that the latent heat of soil moisture is the only heat that needs to be conducted to or from a point which is in the process of thawing or freezing. Heat quantities involved in temperature changes above or below the freezing point are considered to be of minor importance and therefore, ignored. A common form of the Stefan equation for uniform soil conditions is as follows:

$$D = \sqrt{\frac{48k_iF}{L}} \qquad (3\text{-}22)$$

Several investigators (121, 122, 123) have adapted the Stefan equation to layered systems by introducing a concept of thermal resistance and using a trial and error method of solving for the depth of frost penetration.

Since the Stefan equation neglects the heat capacity of the frozen and unfrozen soil, it will give frost penetration depths which are always too large. The degree of overprediction will depend on the climate at the site being studied and the moisture content of the soil.

The modified Berggren equation which was developed by Aldrich and Paynter (124) has found wide acceptance for calculating the depth of frost penetration in soils. Since the basic approach to the mathematical solution is the same as that used by Berggren (125), Aldrich and Paynter (124) named their resulting frost depth prediction equation the modified Berggren equation.

The solution to the modified Berggren equation is based on earlier work by Neumann [see Jumikis (73)] for analyzing freezing and thawing problems in still water. It is assumed that the soil is a semi-infinite mass with uniform properties and that the surface temperature suddenly changes from an initial temperature v_o degrees above freezing to v_s degrees below freezing where it remains constant (Figure 3.36). The general expression for the modified Berggren equation is given as follows:

$$D = \lambda \sqrt{\frac{48 k_i v_s t}{L}} \qquad (3\text{-}23)$$

In Equation 3-23, λ is a function of the freezing index, the mean annual temperature of the site, and the thermal properties of the soil. It is computed from the thermal ratio, $\bar{\alpha}$, and the fusion parameter, $\bar{\mu}$. The thermal ratio, $\bar{\alpha}$, measures the ratio of heat stored initially in the unfrozen soil to the heat loss in the frozen soil. More detailed discussion of the stefan equation and modified Berggren equation can be found in work by Dempsey (9, 17).

The advent of the digital computer has created interest in numerical methods for solving transient heat flow in pavement systems. Although the numerical procedures are approximations of the Fourier diffusion equation (Equation 3-21), they are normally more accurate in solving transient heat flow problems than most of the analytical techniques discussed previously.

Computer programs using numerical heat transfer methods have been written which allow for a variety of initial conditions and boundary conditions.

Table 3.1, which was prepared by Berg (19), shows a summary of most of the numerical methods which have been applied to heat transfer problems in soil-water systems. Table 3.1 shows that explicit and implicit finite difference programs as well as finite element programs have been used to solve heat transfer problems. Table 3.1 also shows that numerical methods can be programmed to consider many different soil conditions, boundary conditions, and soil thermal properties. Both one-dimensional and two-dimensional heat transfer can be determined by use of the numerical methods available.

Dempsey (9, 17) has discussed many of the heat transfer models presently in use for pavement temperature prediction. The application of heat-transfer techniques to the investigation of temperature effects in pavement systems can be found in investigations by Dempsey (9, 17), Berg (19), Dempsey and Marek (70), Dempsey and Thompson (126), and Christison and Anderson (127).

3.4.4 SIMULTANEOUS MOISTURE AND HEAT FLOW

The flow equations which have been considered previously do not include the influence of solutes or temperature gradients on moisture movement. Hillel (128) has discussed simultaneous water and solute movement in soils and this area will not be considered further herein. However, the simultaneous movement of heat and water is a common occurrence and very important to the development of pavement design methodologies to resist the influence of climatic effects.

The fact that temperature gradients can induce water movement in soils has been known for at least 50 yrs (129). Studies on the relative importance and interaction of thermal and suction gradients in transporting soil moisture

have been carried out by Hutchinson, Dixon, and Denbigh (130), Philip and de Vries (131, Taylor and Cary (132, 133), Cassel (134), Cary (135), Hoekstra (136), and Jimikis (137).

From a mechanistic approach, Philip and de Vries (131) developed the following equation for water movement under a combined moisture and temperature gradient:

$$Q = D_{\theta}\nabla\theta + D_{T}\nabla T + K(\theta) \tag{3-24}$$

In Equation 3-24, Q is the net water flux, D_{θ} is the isothermal moisture diffusivity, $\nabla\theta$ is the moisture content gradient, D_T is the thermal moisture diffusivity, and ∇T is the temperature gradient. The terms D_{θ} and D_T are made up of two components each, one for vapor flow and one for liquid flow. The term $K(\theta)$ is the gravity flow term.

Cassel (135) and Dirksen (139) have compared experimental results to the predictions by the Philip and de Vries model. Dempsey and Elzeftawy (10, 100), Dempsey (139), and Elzeftawy and Dempsey (140) have discussed in detail the results of research on modeling simultaneous moisture and heat flow in pavement systems. Figures 3.37 and 3.38 show the results of uniform temperature and a temperature gradient respectively on moisture movement in a subgrade soil above a water table.

In general, the temperature of the soil influences water transport through its effect on the forces that cause the water to move, and through its effect on the conductivities and diffusivities in the various flux equations. In any given application of flow theory, the significance of the water transport due to nonisothermal conditions must be assessed. Water flow in response to a temperature gradient is usually in the direction of

decreasing temperature. In porous media with a gas phase, a temperature gradient produces an associated vapor pressure gradient and a surface tension gradient. There is then a component of the vapor flux that can be related to the applied temperature gradient, and a component of the liquid flux which is associated with the temperature gradient.

3.4.5 WATER FLOW PARAMETERS

The physical soil properties of importance in drainage design or in drainage research and investigations are the saturated hydraulic conductivity, K, the unsaturated hydraulic conductivity, $K(\theta)$, the soil-water pressure head, H, the water content (volumetric, θ, or gravimetric, w), the water-table position, and the degree of saturation, S_r. Bouwer and Jackson (141) have reviewed many of the methods for determining the saturated and unsaturated hydraulic conductivities of soils and materials.

3.4.5.1 SATURATED HYDRAULIC CONDUCTIVITY

The method of measuring the hydraulic conductivity for drainage design should in principle be selected so that the soil region and flow direction for the measurement adequately represent the soil and flow direction to the actual drainage systems. However, it is often difficult to completely represent field conditions and hydraulic conductivity values are generally obtained from laboratory experiments or from in-situ field measurements on a relatively small region of soil.

The normal procedures for measuring hydraulic conductivity in the laboratory are by using a constant-head permeameter or a falling-head permeameter (5, 93). Bouwer and Jackson (141) have discussed procedures for measuring hydraulic conductivity in-situ for positions below the water table and above the water table. For hydraulic conductivity measurements below the water table the following methods are recommended.

1. Auger-hole method: The auger-hole method was first applied by Diserens (142). Various forms of the auger-hole method have been developed by Hooghoudt (143), Van Bauel and Kirkham (144), and Boast and Kirkham (145). The auger-hole method is among the most widely used methods for measuring hydraulic conductivity for agricultural drainage design. In this method the hydraulic conductivity is measured mostly in the horizontal direction.

2. Tube Method: The tube method utilizes an auger hole with the wall lined with a solid tube. The tube does not penetrate the hole bottom. The method is suitable for unstable soils and also for coarse soils if good contact between the tube and hole wall can be obtained.
3. Piezometer method: The piezometer method has been discussed by Luthin and Kirkham (146). The piezometer method measures the hydraulic conductivity in the horizontal or vertical direction depending upon the cavity space below the piezometer. The procedure is not suitable for granular soils.
4. Well Technique: This procedure may consist of the two well techniques proposed by Childs (147), or the multiple well technique by Smiles and Youngs (148). The well techniques measure hydraulic conductivity in essentially the horizontal direction. The hydraulic conductivity is calculated based on the pumping rate and the water level difference in the wells.

For hydraulic conductivity measurements in the absence of a water table Bouwer and Jackson (141) have discussed the following field tests.

1. Shallow well pump-in method: In this method an auger hole is filled with water and a constant water depth is maintained until flow into the soil becomes constant. The method measures K mainly in the horizontal direction. The method is adaptable to coarse soils.

2. Cylinder permeameter method: This technique, developed by Winger (149), is carried out in a fairly large-diameter hole. A steel cylinder with a diameter of about 50 cm is placed in the center of the hole and driven about 15 cm into the soil. Tensiometers are installed in the annular space between the cylinder and the hole periphery with the ceramic cups about 20 cm below the hole bottom. The soil inside the cylinder is covered with a layer of sand, and water is applied to the cylinder and the rest of the hole to maintain equal water depth of about 15 cm inside and outside the cylinder.

   When the tensiometers indicate zero pressure head, saturation is assumed. The value of K is then calculated with Darcy's equation, whereby the gradient is evaluated from the water depth in the cylinder and the depth and pressure head of the tensiometers. The cylinder permeameter measures K in vertical direction. The technique is not suitable for granular soils.

3. Infiltration gradient technique: The cylinder permeameter method was modified by Bouwer (150), who used two concentric tubes in an auger hole and small, fast-reacting piezometers which were pushed down by increments to obtain the complete vertical hydraulic gradient. With this technique, vertical flow in the soil below

the hole bottom could be ascertained and the effect of surface sealing of the soil on the measured value of K could be eliminated. The infiltration gradient technique measures K in a vertical direction. The technique is not suitable for use in soils containing gravel.

4. Air-Entry permeameter technique: With the air-entry permeameter, K is calculated with Darcy's equation from the infiltration rate under high head and a gradient that is measured indirectly without the use of piezometers or tensiometers in the soil. The air-entry permeameter measures K in the vertical direction. The method can be used for gravel soils if the soil can be packed against the sides of the test cylinder.
5. Double-Tube method: The double-tube method developed by Bouwer (151, 152) utilizes two concentric tubes in an auger hole. The soil below the auger hole is wetted and K is evaluated from a flow system created in the soil by letting the pressure head in the inner tube become less than that in the outer tube. The value of K from the double-tube method is affected by vertical and horizontal conductivity, but it is closer to K in the vertical direction. The method is not suitable for granular soils.

Numerous methods have been developed for measuring in-situ permeability of pavement materials and soils. Maytin (153) has developed a field test for determining the permeability through shoulder materials. Moulton (154) is presently working on an FHWA project with the objective of developing a test method and apparatus to determine in-situ permeability of base, subbase, and subgrade materials. Work on permeability measurements of soil-aggregate bases and subbases is also being accomplished in New Jersey (155).

Moulton (14) has developed a procedure for determining the coefficient of permeability for granular materials based on those properties known to influence permeability (Figure 3.39). Figure 3.39 was developed from correlating statistically the coefficient of permeability for a large number of samples. The significant properties considered were the effective grain size, percentage of material passing the No. 200 sieve, and the porosity.

The saturated hydraulic conductivity or coefficient of permeability must be determined when evaluating materials for pavement subdrainage.

3.4.5.2 UNSATURATED HYDRAULIC CONDUCTIVITY

Most of the steady-state methods for measuring the unsaturated hydraulic conductivity, $K(\theta)$, are variations of the long column method or two-plate technique. Transient-state techniques have been developed by Gardner (156), Bruce and Klute (157), and Watson (158). Values of $K(\theta)$ based on pore size data and soil-water characteristics have been determined by Marshall (159), Millington and Quirk (160) Green and Corey (161), Warrick, Mullen, and Nielsen (162, 163), and Elzeftawy and Dempsey (97, 98).

Methods of obtained $K(\theta)$ can be listed as follows:

1. Two-plate method.
2. Long-column method.
3. Advance of the wetting front method.
4. Pressure plate outflow method.
5. Instantaneous profile method.
6. Entrapped air method.
7. Theoretical methods based on soil properties and soil-water characteristics.

Dempsey and Elzeftawy (10, 100) have used the instantaneous profile method to determine $K(\theta)$ in their research on unsaturated moisture movement in pavement subgrade soils.

Investigations by Elzeftawy and Dempsey (10, 100) have been completed to develop laboratory procedures for determining $K(\theta)$ for fine grained and

coarse grained soils in the laboratory. Dempsey is present completing, a research project sponsored by the Illinois Department of Transportation, to determine both saturated and unsaturated hydraulic conductivities for major Illinois soil types.

3.4.6 HEAT-FLOW PARAMETERS

Dempsey (9, 17) has discussed at length the extrinsic and intrinsic parameters involved in heat-transfer in pavement systems.

The extrinsic factors shown in Figure 3.40 indicate that a broad interpretation of climate is necessary when considering its influence on temperature and temperature effects in pavement systems. Dempsey (9, 17) has indicated that the climatic factors influencing pavement temperatures can be categorized as follows:

1. Temperature factors, which directly affect the transfer of heat to or from the pavement surface, such as air temperature, shortwave solar radiation, long-wave radiation, and wind.
2. Hydrologic factors, which exert an indirect influence on the temperature of the pavement system, such as precipitation, evaporation, and condensation.
3. Geographical factors, which exert a direct influence on weather and its outcome, such as elevation, latitude, degree of exposure, and nearness to bodies of water.

The temperature factors related to the pavement surface are among the most important input parameters in pavement temperature prediction and moisture movement These factors are concerned with the net radiation heat transfer, and the convective heat transfer into or out of the pavement system.

Dempsey (9, 17) has discussed in detail most of the intrinsic factors which influence frost action and temperature effects in pavement systems

(Figure 3.41). It is generally agreed that the most important intrinsic factors required in a heat transfer model are the thermal properties of the pavement materials, which include thermal conductivity, heat capacity, and latent heat of fusion.

The thermal conductivity, k, is the quantity of heat which flows normally across a surface of unit area per unit time under a unit thermal gradient. The most common units of thermal conductivity are Btu/hr-ft-°F, Btu/hr-ft^2-°F/in., or Cal/sec-cm-°C. Experimental measurements of thermal conductivity can be accomplished by several methods, all of which are based on the observation of the temperature gradient across a given area of the material conducting heat at a known rate.

The heat capacity, c, is the amount of thermal energy necessary to cause a 1-deg temperature change in a unit mass or unit volume of substance. The units of heat capacity depend on whether mass heat capacity or volumetric heat capacity is used. Mass heat capacity has the units of Cal/g-°C or Btu/lb-°F. The units of volumetric heat capacity are Btu/ft^3-°F or Cal/cm^3-°C. Generally the heat capacity is computed from a heat balance between the heat gained by water in a calorimeter and the heat gained by the specimen.

The latent heat of fusion, L, is the change in thermal energy in a unit volume of material when the moisture in that material freezes or thaws at a constant temperature. The common units for the latent heat of fusion of soil moisture are Btu/ft^3 or Cal/cm^3. The latent heat depends upon the percentage of water which can be frozen in a given volume of material at any given temperature.

In recent years, the thermal properties of soils have been the subject of numerous laboratory and field studies. However, very little effort has been directed at developing thermal values for paving materials such as portland cement concrete and bituminous concrete.

Most of the literature concerned with the thermal properties of pavement surface materials are found in general tables of physical properties for building materials or are shown in scientific research.

Thermal conductivity data for granular materials and soils have been developed by Moulton and Dubble (32), Jimikis (73), Kersten (164), McGaw (165), Mickley (166), and Gemant (167). It has generally been indicated that the procedures proposed by Kersten (164) are adequate for most heat-transfer investigations in pavement systems.

3.5 SUBSURFACE DRAINAGE MATERIALS AND EQUIPMENT

3.5.1 DRAINAGE MATERIALS

Clay and concrete drain tile have been the principal subdrainage materials for many decades. Fouss (168) has indicated that polyethylene plastic was made available for manufacture in the United States in about 1941. He indicated that the Corps of Engineers investigated the use of perforated plastic tubing for airport drainage in 1946. Considerable changes and modernization in drainage for both agricultural and engineering purposes took place during the period from 1960 to 1970.

Presently several different types of drainage pipe of various lengths and diameters are being used in pavement subsurface drainage. Some of these are as follows:

1. Clay tile.
2. Concrete tile and pipe.
3. Vitrified clay pipe.

4. Perforated plastic and bituminous fiber pipe.

5. Perforated corregated-metal pipe.

6. Corrugated plastic tubing.

The clay and concrete tile can be obtained in 0- to 3-ft lengths. Metal and fiber pipes are usually manufactured in lengths of 8 ft or longer. The thick-walled, semi-rigid plastic tubing may be obtained in about 20-ft lengths. The corrugated plastic tubing is manufactured in rolls about 200- to 300-ft long. For subsurface drainage the pipe diameter generally ranges between 4 in. and 6 in. However the California Department of Transportation uses slotted plastic pipe with an inside diameter of 1-1/2 in.

In Project 4-11 of the National Cooperative Highway Research Program research is under way to develop and evaluate the design, installation, and performance criteria for the use of buried plastic pipe in transportation facilities (169). This project is evaluating field tests in Illinois, Maine, and New Hampshire. Illinois has installed perforated plastic and bituminous fiber pipe as well as corrugated plastic tubing. The Maine study is concerned with plastic pipe buried as culverts at shallow depths. In New Hampshire plastic pipe of various wall thicknesses are being evaluated.

Most of the newer drainage pipe are flexible conduits rather than rigid conduits such as clay, concrete, or metal conduit. Failure in flexible plastic subdrains is usually a result of excessive deflection. For this reason the load-deflection characteristics are important considerations when this material is being used in subsurface drainage design. Impact resistance is also important from the standpoint of damage to the pipe while it is being placed during construction. NCHRP Project 4-11 has discussed many of the standards for evaluating plastic pipe and corrugated plastic tubing. Most of the strength tests are based on ring compression theory (169).

When considering transverse drains, longitudinal drains, drainage blankets, and drainage wells it is necessary to evaluate the filter or envelope material. The primary reasons for placing envelope material around subsurface drains are as follows:

1. To prevent the movement into the drains of soil particles which might settle and clog the drain.
2. To provide material in the immediate vicinity of the drain openings which is more permeable than the surrounding soil.
3. To provide a suitable bedding for the drain.
4. To stabilize the soil on which the drain is being laid.

In agricultural drainage the drainage envelope is designed to allow microscopic suspended material in the soil water to pass through the envelope to the drain (168). If this is not done, the drainage envelope will soon become filled with fine soil particles and the permeability will be reduced. For this reason porous material placed around a subsurface drain should be referred to as an envelope and not as a filter.

Willardson (170) has classified materials for drainage envelopes into organic materials, inorganic materials, and man-made materials. Organic materials such as straw, sawdust, and coconut fiber have been used as filter material in agricultural drainage.

The most common and widely used envelope materials are naturally graded coarse sands and gravels. There is a considerable range of gradations used for drainage envelopes. Slaughter (3) found that the recommended gradations could vary considerably from one state to another. Figures 3.42 and 3.43 show a comparison of the range that can be found between two different state transportation departments. The general procedure for designing a

drainage envelope for a given soil is to make a mechanical analysis of both the soil and the proposed envelope material, compare the two particle size distribution curves, and decide by some criteria whether the envelope material is satisfactory. Willardson (170) has reviewed most of the criteria for drainage envelopes. The criteria proposed by Terzaghi and Peck (93) are as follows:

1. The particle diameter of the 15 percent size of the filter material should be at least 4 times as large as the diameter of the 15 percent size of the base material. This would make the filter material roughly more than 10 times as pervious as the base material.
2. The 15 percent size of the filter material should not be more than 4 times as large as the 85 percent size of the base material. This would prevent the fine particles of the base material from washing through the filter material.

The 15 percent size is the particle diameter such that 15 percent of the material by weight is of a smaller diameter and 85 percent of the material is of a smaller diameter than the 85 percent size.

Juusela (171) reported that specifications for envelope gravel in Finland also required that the 15 percent size of the envelope be at least 4 times the 15 percent size of the soil to assure adequate permeability, and further specifies that the 15 percent size of the envelope should not be more than 3 times the 85 percent size of the soil to prevent movement of fines into the drain.

Bertram (172) conducted tests on natural sands and Ottawa sand, on crushed quartz, and on crushed quartz and Ottawa sand to determine the critical ratios of the 15, 85, and 50 percent sizes of base soil and

envelope material. For single-sized envelope and base materials at least 50 percent compacted, he concluded the following:

1. The minimum critical ratio of the 15 percent size of the envelope to the 15 percent size of the soil at the limit of stability is approximately 9. The minimum critical ratio of the 15 percent size of the envelope to the 85 percent size of the base soil at the limit of stability is approximately 6.
2. The critical ratios are practically independent of the shape of the grains.
3. The critical ratios are fairly constant for the range of hydraulic gradients from 6 to 20 which were investigated.

Using a graded-base material and uniform envelope material, Bertram (172) found the critical ratios to be 6 for the 15 percent size of the envelope to the 85 percent size of the base and 26 for ratio of the 15 percent size of the envelope to the 15 percent size of the base. The ratios changed somewhat for other gradings of natural materials.

Differences in the critical ratios of the 15 and 85 percent sizes recommended by the various researchers change the relative coarseness of the drain envelope. Higher critical ratios allow coarser envelope material.

An investigation of the envelope requirements of underdrains by the Corps of Engineers (173), using laboratory and tank tests, led to conclusions not greatly different from the criteria set by Terzaghi and Peck (93). In addition to the size specifications, it was found that the grain size curves for filter and base materials should be approximately parallel in order to minimize washing of the fine base material into the filter material.

Karpoff (174) of the Bureau of Reclamation published the results of extensive laboratory tests of design criteria for protective filters for hydraulic structures. His conclusions were the following:

1. For uniform grain-size filters consisting of natural subrounded particles the ratio of the 50 percent grain size of filter material to the 50 percent grain size of the base material should be between 5 and 10.
2. For graded filters consisting of natural subrounded particles the ratio of the 50 percent grain size of filter material to the 50 percent grain size of the base material should be between 12 and 58 and the ratio of the 15 percent grain size of filter material to the 15 percent grain size of base materials should be between 12 and 40.
3. For tentative relationships of graded filters consisting of crushed rock the ratio of the 50 percent grain size of filter material to the 50 percent grain size of the base material should be between 9 and 30 and the ratio of the 15 percent grain size of filter material to the 15 percent grain size of base material should be between 6 and 18.
4. In addition to the limiting ratios established for adequate filter design, the following requirements should be met:
    a. The filter material should pass the 8-cm screen for minimizing particle segregation and bridging during placement. Also, filters must not contain more than 5 percent of material sizes smaller than 0.07 mm to prevent excessive movement of fines in the filter and into drainage pipes.

b. The gradation curves of the filter and the base material should be approximately parallel in the range of finer sizes, because the stability and proper function of protective filters depends upon skewness of the gradation curve of the filter toward the fines, in the base material.

c. The filter material adjacent to the drainage pipe should be sufficiently coarse to prevent movement of filter material into the drainage pipe openings. The maximum size of perforations or joint openings of the drainage pipe was selected as one-half of the 85 percent grain size of the filter material. This criterion proved satisfactory in all filter tests conducted in the laboratory.

d. In designing of filters for base materials containing particles larger than a No. 4 sieve size (4.76 mm diameter), the base material should be analyzed on the basis of the gradation of the material smaller than the No. 4 size sieves.

The described criteria established by Karpoff (175) for designing protective filters are used by the Bureau of Reclamation in design of inverted filters for canal slope protection and drainage systems under concrete canal linings, canal structures, and spillways.

Based on the work of Bertram and the Corps of Engineers Cedergren (5) has recommended the following criteria for filter design:

$$\frac{D_{15}\text{ (of filter)}}{5} \leqq D_{85}\text{ (of soil)} \qquad (3\text{-}25)$$

In order to insure reasonable uniformity of gradation and to prevent the use of gap-graded materials Cedergren (5) recommended the following additional criteria:

$$\frac{D_{50}\text{ (of filter)}}{25} \leqq D_{50}\text{ (of soil)} \qquad (3\text{-}26)$$

The criteria set by Cedergren (5) is in agreement with that recommended by Barksdale and Hicks (13).

In the Highway Drainage Manual Moulton (14) adapted the following criteria for the design of protective granular filters:

$$(D_{15}) \text{ filter} \leq 5\ (D_{85}) \text{ protected soil} \tag{3-27}$$

$$(D_{15}) \text{ filter} \geq 5\ (D_{15}) \text{ protected soil} \tag{3-28}$$

$$(D_{50}) \text{ filter} \leq 25\ (D_{50}) \text{ protected soil} \tag{3-29}$$

$$(D_{5}) \text{ filter} \geq 0.074 \text{ mm} \tag{3-30}$$

$$(CU) \text{ filter} = \frac{(D_{60}) \text{ filter}}{(D_{10}) \text{ filter}} \leq 20 \tag{3-34}$$

Other filter criteria have been established by Kruse (175), Pillsbury (176), and the Soil Conservation Service (177).

Winger and Ryan (178) have indicated that drainage designed by the Terzaghi and Peck (93) criteria did not always perform satisfactorily in subsurface drainage work. They felt that the drainage criteria developed for toe drains for dams were based on the assumption of relatively high pressure gradients (7 to 45) which are not present in relatively shallow agricultural drainage where the pressure gradient may be very close to one. For this reason they felt that the drainage envelopes based on the Terzaghi and Peck (93) criteria were too restrictive to agricultural drainage. Table 3.2 shows the recommended gradation relationship between the base material and envelope recommended by Winger and Ryan (178).

Luthin, Taylor, and Prieto (179) conducted a theoretical study of exit gradients into circular drain pipes of various outer diameters. Results of their analysis indicated that the exit gradients into completely permeable drains exceeded the critical gradient for nearly every condition studied. They found that a gravel envelope increased the apparent diameter of the drain and therefore substantially decreased the exit gradient. One of the functions of the gravel envelope around a drain is to prevent high gradients in the soil near drains from causing soil to be carried into the drain.

Luthin and Haig (180) have found that by increasing the effective diameter of the drain pipe by means of a granular envelope could increase the flow by as much as 30 to 40 percent.

Based on work by Elzeftawy and Dempsey (97, 98), and unpublished work by Ring (181) it is important to note that even though some materials have a high saturated hydraulic conductivity they may not drain freely. Figure 3.35 shows the water content in a sand often used as a drainage envelope as a function of height above a water table. Calculations show that the sand is about 90 percent saturated at a point 2 ft above the water table. This percentage of saturation can be very detrimental in areas with freeze-thaw cycles.

Similarly work by Ring, Table 3.3, using various combinations of sand and gravel show that the degree of saturation after free drainage can be almost 95 percent in the fine material as compared to 8 percent in the coarse material. The data in Figure 3.35 and Table 3.3 point out the importance of the drainability of filter materials, especially, in areas where frost action could be a problem.

Based on work by Cedergren (5), FHWA (6), Barksdale and Hicks (13), and Spalding (182) holes and slots in perforated drainage pipe should meet the following criteria:

1. D_{85} (filter) > Diameter of holes.
2. D_{85} (filter) > 1.2 times slot width.

Moulton (14) adopted the following criteria in the Highway Drainage Manual for filter and perforated pipe compatibility:

1. (D_{85}) filter > 1/2 times slot width
2. (D_{85}) filter > 1 times hole diameter

Man-made materials which have received attention as filter materials include fiberglass, nylon, polypropylene nylon materials, and other synthetic materials. Many of these filter fabrics are marketed under the trade names of Drain Guard, Mirafi, and Typar.

The Alabama Highway Department is conducting a study on the use of filter fabric in trench drains on Interstate 65 in Chilton County. Barenberg and Tayabji (7) investigated the use of filter fabric between an open graded bituminous aggregate mixture and the subgrade. The filter fabric did not perform well in this particular test.

Barenberg, Dowland, and Hales (183) have evaluated Mirafi fabric in combination with soil aggregate systems. It was found that the fabric increased the strength of the soil aggregate system.

Hermsmeier (184) has conducted laboratory drainage studies using gravel and various synthetic materials as the drainage envelope. Table 3.4 shows the results of the drainage test for four cycles of drainage. It is noted that the flow rate in all of the sections decreased after the first cycle of drainage. No sediment was noted or measured in the outflow or in the drains during the test. McKyes and Broughton (185) and Broughton, Damont, and English (186) obtained trends similar to those of Hermsmeier (184) in their tests of filtered drain tubes. Figures 3.44 and 3.45 and Table 3.5 show the results of their test. From these tests it would appear that the evaluation of water flow through filters should be extended for a considerable period of time in order to determine any significant flow decrease with time.

The general filter requirements for drainage envelopes also apply to the design of filter protection of open-graded drainage layers. A more detailed discussion of this area can be found in Cedergren (5).

Cedergren (5) has indicated that the pipe outlets for surface drainage should be located so that they drain freely into collector pipes, ditches, or some other type of drainage facility. In cold regions the outlet pipes should be prevented from freezing. In order to facilitate maintenance operations the pipe outlets should be clearly marked.

3.5.2 EQUIPMENT

The equipment for installation of subsurface drainage has been described by Fouss (168) and others (187). In agricultural work subsurface drainage is primarily installed by trenchers or plow-type equipment. The trenchers used in pavement subdrainage work are normally the same, with some modifications, as those used in agricultural drainage. Grade control in pavement subdrainage work is generally obtained from the mainline pavement section. However, modern drainage installation equipment can be easily equipped with automatic grade control systems.

3.6 EXISTING DESIGN PRACTICES

For the analysis and design of drainage for pavement structural sections the procedure proposed by Moulton (14) is recommended. This procedure involves the following steps:

1. Assemble all available data on highway and subsurface geometry, index properties and performance characteristics of soils and materials, precipitation and frost penetration, and miscellaneous considerations.
2. Determine the net inflow, or quantity of water, that must be removed by the pavement drainage system. The gross inflow would consist of water from all sources that might contribute to the possible saturation of the pavement section under consideration, including groundwater infiltration and melt water from thawing ice lenses (where frost action is present) in the subgrade soil. In computing the net inflow, an allowance should be made for any natural outflow that can take place by vertical seepage into the soil beneath the pavement.
3. Analyze and/or design pavement drainage layer(s) to provide for the rapid removal of the net inflow determined in Step 2. This

should include an evaluation of the need for filter layers or special treatment of the subgrade.

4. Analyze and/or design collection system(s) to provide for the disposal of water removed by the pavement drainage layers. This includes the location and sizing of longitudinal and transverse collector drains, selection of filter material, and determination of outlet spacing.

5. Conduct a critical evaluation of the results of Steps 3 and 4 with respect to potential long-term performance, construction, maintenance and economics of the proposed pavement drainage system.

Moulton (14) indicated that steps 3, 4, and 5 are interdependent and that it may be necessary to pursue certain aspects of all three steps simultaneously or in some order other than that in which they were presented. He indicated that the thickness of the required drainage layers is governed, in part, by the distance the water has to flow to reach an outlet. This distance is controlled by the type and geometry of the selected collection system, which in turn may be governed by the economics associated with the cost and availability of granular drainage materials and pipe.

Barksdale and Hicks (13) stated that design of the subsurface drainage system should be approached in a rational manner using the fundamental principals of hydrology and the flow of water through porous materials. They indicated that the following factors influenced the time required for water to drain from beneath a pavement structural section:

1. Rainfall intensity and duration of the selected design storm.
2. Percentage of the total rainfall entering the pavement.
3. The quantity of groundwater seepage reaching the pavement.
4. Permeability and thickness of the relatively free draining layers and the permeability of the subgrade.

5. Longitudinal and transverse slope of the drainage layers.
6. Width of the pavement structure.
7. Construction variables which can influence the performance of the drainage system.

Barksdale and Hicks (13) have stated that pavement shoulders in areas having as low as 20 to 30 in. of rainfall each year can undergo significant structural damage caused by surface and subsurface water.

Cedergren (5) and the FHWA (6) have stated that large accumulations of free water must be prevented in the structural sections of pavements carrying heavy wheel loads. They suggest that all possible inflows and all beneficial outflows be related in the following mathematical form

$$\Sigma O \geqq \Sigma I \tag{3-32}$$

In Equation 3-32 ΣI represents all inflow sources and ΣO represents all outflow possibilities. In designing subsurface drainage Cedergren (5) recommends a factor of safety, C, in estimating the needs to be removed so Equation 3-32 becomes:

$$\Sigma O \geqq C\Sigma I \tag{3-33}$$

In using the inflow-outflow procedure Cedergren (5) recommends a concept called the condition of continuity where the outflow capabilities of a system should increase in the direction of flow, starting at the point of entry and progressing through the base drainage layer, collector pipes, and outlet pipes. Figure 3.46 shows the hypotehtical flow path of water entering through a pavement (A - B), flowing downward to a base drainage layer (B - C), laterally out of the base drainage layer (C - D), into the collector pipes (D - E), and through the outlet pipe to the point of discharge (E-F). The flow throughout the system is estimated by Darcy's Law or other analytical or experimental procedure. In order to prevent water from accumulating

in the system Cedergren (5) recommends that the following equation be satisfied:

$$Q_{A-B} \leqq Q_{B-C} \leqq Q_{C-D} \leqq Q_{D-E} \leqq Q_{E-F} \qquad (3\text{-}34)$$

He indicates that Equation 3-34 provides a useful criteria for checking the adequacy of each of the component parts of the flow system.

Cedergren (5) and the FHWA (6) have made the following recommendations for determining the surface infiltration:

1. Select the design precipitation rate based on a one-hour rainfall with a frequency of occurrence of 1 year.
2. Determine the design infiltration rate by multiplying the design precipitation rate by a coefficient between 0.50 and 0.67 for concrete pavements and between 0.33 and 0.50 for asphalt concrete pavements.
3. Design the drainage system to have an outflow rate equal to or greater than the inflow rate.

Cedergren (5) and the FHWA (6) indicate that infiltration from capillary water, groundwater flows, and water of hydrogeneses should also be considered in the estimation of inflow rates.

Based on work by Ridgeway (188), the following equation was recommended by Moulton (14) for determining infiltration:

$$q_i = I_c \left[\frac{N_c}{W} + \frac{W_c}{WC_s} \right] + K_p \qquad (3\text{-}35)$$

where

q_i = design infiltration rate,

I_c = crack infiltration rate,

N_c = number of contributing longitudinal cracks,

W = width of granular base or subbase subjected to infiltration,

WC = length of contributing transverse cracks,

C_s = spacing of transverse cracks or joints, and

K_p = infiltration rate through uncracked pavement surface.

Moulton (14) also recommends procedures for estimating the net pavement inflow based on groundwater seepage, melt water from ice lenses, and vertical outflow.

Cedergren (5) stated that outflow rates in the pavement section may be reduced as a result of surface evaporation, lateral seepage away from the pavement, subgrade percolation or drainage, and from pumping through joints and cracks in the pavement. Cedergren (5) indicated that the time required for drainage of the pavement system is important. He recommended that the allowable drainage time in highway pavements should be 1/2 hr in cold regions and 1 hr for all other areas. He indicated that 1 to 2 hr would be a reasonable drainage time in wider airport pavement systems.

Barksdale and Hicks (13) have suggested a more conservative estimate of drainage time. They indicate that drainage systems which can remove 50 percent of the available free water from the structural pavement section within 2 to 6 hr after saturation should be adequate. They did state that more rapid removal of the water might be justified in area with frost action or expansive soils. By considering various geometrical parameters and pavement hydraulic and engineering properties Barksdale and Hicks (13) have developed a method for determining the time required for 50 percent of the total free water to be drained from the pavement structural layer as follows:

$$t_{50} = \frac{c\ n_e\ L_e^2}{2KH}\ T_{50} \tag{3-35}$$

where

t_{50} = time for 50 percent of the total free water to be drained.

c = A factor based on pavement slope.

n_e = The effective porosity of the drainage layer.

L_e = Effective length of the drainage path.

K = Saturated hydraulic conductivity.

H = Thickness of the drainage layer.

T_{50} = Dimensionless time factor.

The Corps of Engineers (189) have also presented a formula for calculating the time required for 50 percent drainage to occur as follows:

$$t = \frac{n_e D^2}{2880\ KH_o} \qquad (3\text{-}36)$$

In Equation 3-36, t is the time for 50 percent drainage to occur, n_e is the effective porosity, D is the sloping width, K is the hydraulic conductivity in feet per minute, and H_o is a function of the base thickness and cross slope. The FHWA (6) has recommended the use of open graded bases with a filter layer to prevent the intrusion of subgrade soil into subsurface drainage systems. Figure 3.47 shows typical gradations and permeabilities of open graded bases and filter materials. Figure 3.47 shows the levels of permeability that are possible for the range of material gradations.

Collector pipes are used to expedite the removal of water from the drainage layers. These pipes are placed in shallow V trenches or rectangular trenches. Figure 3.48 shows typical cross sections of subdrainage systems. As noted in Figure 3.48 the depth and location of the collector pipes and outlet pipes can vary considerably. Figure 3.49 shows the location of transverse drains in a typical pavement section. Table 3.6 shows the drainage practices of several states. Figure 3.50 is an improved pavement-shoulder drainage system used in Georgia. Similarly Figure 3.51 shows the drainage system presently being used by the Illinois Department of Transportation. The FHWA (6) has indicated that pipe outlets should be located based on design considerations. Figure 3.52 presents a nomograph for selecting the diameter of perforated pipe and the spacing of outlets. An outlet is recommended at the sag of vertical curves.

The open end of the outlet pipe should be at least 12 in. above the flow line of the roadside ditch and protected from damage and intrusion of foreign matter. A splash block or headwall is recommended at the pipe outlet. The outlet pipe should be backfilled with material of low permeability in order to prevent piping. Marker post are recommended at each collector pipe outlet.

The FHWA (6) has indicated that criteria for areas where subsurface drainage systems may not be required are as follows:

1. Where the average normal precipitation is less than 10 in.
2. When the lateral drainage transmissibility of the base layer beneath the highway pavement surface is 100 times greater that the design infiltration rate.
3. When the combined lateral drainage transmissibility of the base, and the vertical drainage capability of the underlying materials exceed the design infiltration rate.
4. When 250 or less 18,000-lb.axial loads per day are predicted during the design life of rigid pavement systems.

The FHWA (6) indicated that some adjustments in the above criteria may be necessary in areas where frost action is present.

Detailed discussion of the methods for drainage analysis and design developed up to this point in time are presented by Cedergren (5), FHWA (6), Barksdale and Hicks (13), Moulton (14), and the Asphalt Institute (190).

3.7 SUMMARY

Considerable progress has been made into the analysis and design of subdrainage for pavement systems. However, there are numerous areas which still need to be investigated and incorporated into the design procedure. The hydrological considerations in subdrainage analysis and design need to be defined more clearly. The importance of the hydraulic properties of the pavement materials, subgrade, and drainage materials should be more fully understood and appreciated.

Considerable work is still needed in the proper design of envelope materials around the drainage pipe. The use of filter fabrics also need to be investigated.

Proper recognition of pavement systems which can benefit from subsurface drainage is a critical need and an important part of this study. The cost effectiveness of subsurface drainage as it relates to the long term performance of pavement systems needs to be established.

Investigations of numerous pavements as part of the FHWA project on "Zero Maintenance Pavement Systems" indicated that water was one of the major factors contributing to distress and early serviceability loss. Based on this study it is evident that hydrothermal design practices need to be developed in conjunction with design based on load.

Table 3.1

Numerical Methods Applied to Heat Transfer in Soil-Water System (19).

		Type			Dimension				Soil				Boundary Conditions							Init. Conditions		Thermal Properties						Heat Transfer Mechanisn					Type of Solution			
		Finite Diff.							Iso-tropic				Upper				Lower											Mass Flux								
Name	Date	Explicit	Implicit	Finite Element	One-Dimensional	Radial	Two-Dimensional	Three-Dimensional	Homogeneous	Layered	Non-Homogeneous	Anisotropic	Air Temperature	Programmed Temp.	Surface Heat Flux	Constant Temp.	Periodic Temp.	Variable Temp.	Heat Flux.	Constant Temp.	Specified Temp.	Latent Heat	Unfrozen Moisture	k Constant w/Temp.	k Varies w/Temp.	c Constant w/Temp.	c Varies w/Temp.	Vapor	Vertical Liquid	Hor. Liquid	Conduction	Radiation	Temp. Profile	32°F Isotherm	Tabular	Graphical
Hashezi & Sliepcevich	1965		X		X	X	X		X	X			X		X			X	X		X	X	X		X		X			X	X		X	X		X
Carroll, Schenck, & Williams	1966	X			X				X	X			X		X	X					X	X			X		X				X		X		X	
Chwang	1967	X					X		X	X	X		X	X		X					X	X			X		X				X				X	
Dempsey & Thompson	1969	X			X				X	X			X		X	X					X	X			X		X				X		X	X	X	
Ho	1969		X		X		X		X	X			X	X		X					X	X	X		X		X				X		X	X	X	
Doherty	1970	X					X		X					X		X					X	X			X						X		X		X	
McDonnell-Douglas	1971	X			X	X	X		X	X	X		X	X	X	X					X	X	X		X		X				X		X	X	X	X
Makano & Brown	1971		X		X				X	X			X	X		X					X	X	X		X		X				X		X	X	X	
Pal'Krn	1971	X			X	X	X		X	X			X	X		X					X	X			X		X				X		X	X	X	
Berg & McDougall	1971	X			X				X	X			X	X		X					X	X		X		X					X		X	X	X	X
Esso Production	1970		X			X	X		X	X	X		X	X	X	X			X		X	X	X		X		X				X		X	X	X	X
Christison & Anderson	1972	X			X				X	X					X	X					X	X	X		X		X				X		X		X	
Dow Chemical Company	1972	X		X	X	X	X		X	X	X	X	X	X	X	X	X	X	X	X	X	X			X		X				X		X	X	X	
Goodrich	1972	X			X				X	X			X	X				X	X		X	X	X		X		X				X		X	X	X	X
Harlan	1972		X		X				X					X		X				X		X	X		X		X		X		X		X		X	
Hwang, Murray, & Brooker	1972			X	X	X	X		X	X	X		X		X	X			X		X	X	X		X		X				X		X	X	X	
Meyer, Keller, & Couch	1972		X		X				X	X				X		X					X	X			X		X				X			X		X
Mohan	1972			X	X	X	X		X	X	X		X			X					X	X			X		X				X		X	X	X	
Williamson	1972	X			X				X	X			X		X	X					X				X		X				X					

Note: Dow Chemical Company has two programs.

Table 3.2

Gradation Relationship Between Base Material and Envelope (178)

Base material 60 percent passing (diameter of particles, mm)	Graduation limitations for envelope (diameter of particles, mm)											
	Lower limits						Upper limits					
	Percent passing						Percent passing					
	100	60	30	10	5	0	100	60	30	10	5	0
0.02-0.05	9.52	2.0	0.81	0.33	0.3	0.074	38.1	10.0	8.7	2.5	-	0.59
0.05-0.10	9.52	3.0	1.07	0.38	0.3	0.074	38.1	12.0	10.4	3.0	-	0.59
0.10-0.25	9.52	4.0	1.30	0.40	0.3	0.074	38.1	15.0	13.1	3.8	-	0.59
0.25-1.00	9.52	5.0	1.45	0.42	0.3	0.074	38.1	20.0	17.3	5.0	-	0.59

Table 3.3

Water Retention Test (FHWA)

Material	Density (PCF)	Initial Saturation (%)	Saturation After Drainage (%)
Pea Gravel	100.7	99.4	8.2
Sand	103.3	98.0	90.7
Sand	101.7	100.0	94.8
25% Sand + 75% Pea Gravel	113.2	98.6	46.9
25% Sand + 75% Pea Gravel	100.2	100.0	46.3

Table 3.4

Flow Rates From Drains With Different Envelope Materials Installed in Test Tank (184).

	1st Cycle	2nd Cycle	3rd Cycle	4th Cycle
		(ml/min/m)		
Gravel (G)	104.4	27.9	24.4	27.8
Drain Guard (D)	36.6	14.4	9.4	14.1
Typar, 3-1/2 oz. (T)	36.7	20.3	9.4	10.3
Mirafi 140 (M)	38.4	12.8	9.9	8.6

Table 3.5

Summary of Soil Retention Qualities and Water Flow Rates for Drain Tube Filter Materials (185).

Filter Type	Flow Period in Operation	Soil Retention	Stabilized Water Flow Rate (ml/min per 24 cm tube length) Period A Start	Period A End	Period 1 Start	Period 1 End	Period 2 Start	Period 2 End	Period 3 Start	Period 3 End
No filter	A	Unacceptable	250	15						
Fiberglass tile guard	A	Good	115	65						
Spun bonded polyester Remay 0.6 oz/sq yd	A	Good	105	40						
Twine in corrugations	A	Good	150	0						
"Superfilter" (polyester and rayon)	1, 2, 3	Good			125	35	95	29	70	20
Spun bonded nylon "Cirex" 0.6 oz/sq yd	1, 2, 3	Good			115	45	96	29	70	18
Woven nylon yarn stocking	1, 2, 3	Good			90	40	96	29	40	18
Spun bonded polyester Remay 0.8 oz/sq yd	1, 2, 3	Good			110	30	70	21	30	14
Typar spun bonded polypropylene 4 oz/sq yd	2, 3	Good					280	40	290	26
Coconut fiber on 65 mm tubing	2	Poor								
Coconut fiber on 80 mm tubing	2	Poor					190	55		
Typar 1.5 oz/sq yd	3	Poor							162	40
Coarse concrete sand 5.7 cm thick	3	Good							1500	90
First Certainteed-Daymond nylon Stocking	3	Poor							110	40
Second Certainteed-Daymond nylon stocking	3	Good								

Table 3.6

Drainage Practices of Several States (13)

HIGHWAY ORGANIZATION	SURFACE DRAINAGE Pavement Slope	Shoulder Slope	SUBSURFACE DRAINAGE LAYER Material	Thickness	Extent	PIPE SHOULDER DRAIN Type Location	Size (in.)
Arizona	1/4 in/ft	1/4 in/ft	Granular Base or Subbase	4 in. (min.)	Daylighted	None Provided	-
California	1/4 in/ft	$1\frac{1}{2}$ in/ft	Granular Base Beneath Shoulder	5 in. (min.)	Edge of Paved Shoulder	Wet Conditions Only	-
			Asphalt Stabilized Drainage Blanket (exp. only)	5 in. (min.)	Full Shoulder Width	1 in. PVC (exp. only)	
Georgia	1/8 in/ft	1/2 in/ft	-	-	-	Longitudinal Drain Standard Design	
Illinois	1/4 in/ft	1/2 in/ft	Open Graded Aggregate Subbase	4-6 in.	Daylighted	Wet Conditions or During Shoulder Rehabilitation	-
Louisiana	1/4 in/ft	$1\frac{1}{2}$ in/ft	Experimental Only	-	-	Experimental Only	-
Michigan	-	-	Granular Base and Subbase	14 in.	Daylighted	Use Edge Drains Where Drainage Path - 30 Feet	
Minnesota	3/16 in/ft	1 in/ft	Granular Base	6 in.	-	Cuts Only	-
New York	1/4 in/ft	1 in/ft	Granular Base and Subbase	12 in.	Daylighted	Low Points Only	-
North Dakota	1/8 in/ft	1 in/ft	-	-	-	Seldom Used	-
Ohio(1)	3/16 in/ft (reverse slope)	1/2 in/ft	Granular Subbase	$7\frac{1}{2}$ in.	Daylighted	For Subgrade Soils A-4, A-6, A-7-6	6 in. dia. Clay or Metal Pipe
Pennsylvania	1/4 in/ft	1/2 in/ft	Granular Subbase	6-12 in.	Daylighted	18-in. From Pavement Edge	6 in.dia. Clay or Metal Pipe
Texas	1/4 in/ft	3/8 to 1/2 in/ft	Not Normally Used	-	-	Wet Conditions Only	-
Utah	1/4 in/ft	1/4 in/ft	Not Normally Used	-	-	Not Used	-

(1) New pavements will have stabilized base. No provision for drainage.

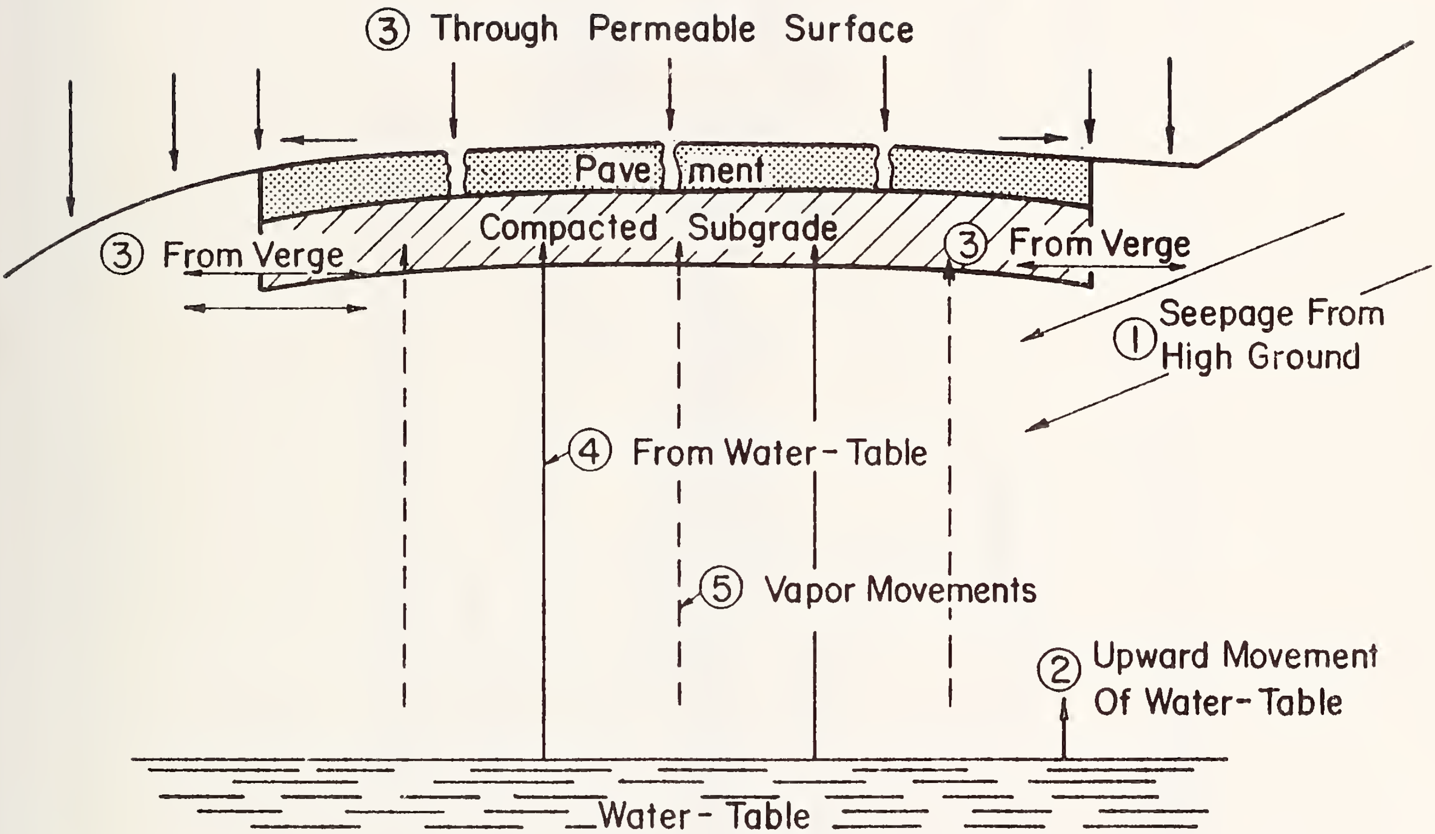


Figure 3.1 Sources of Moisture in Pavement Systems (21).

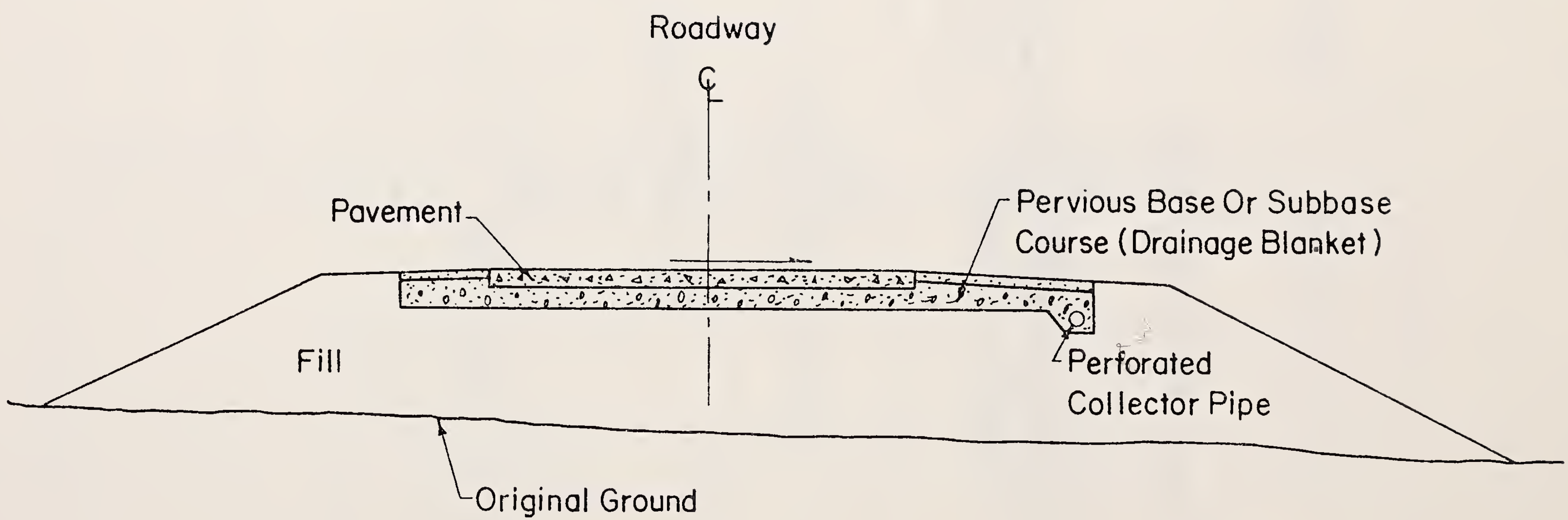


Figure 3.2. Longitudinal Collector Drain Used to Remove Water Seeping Into Pavement Structural Section (14).

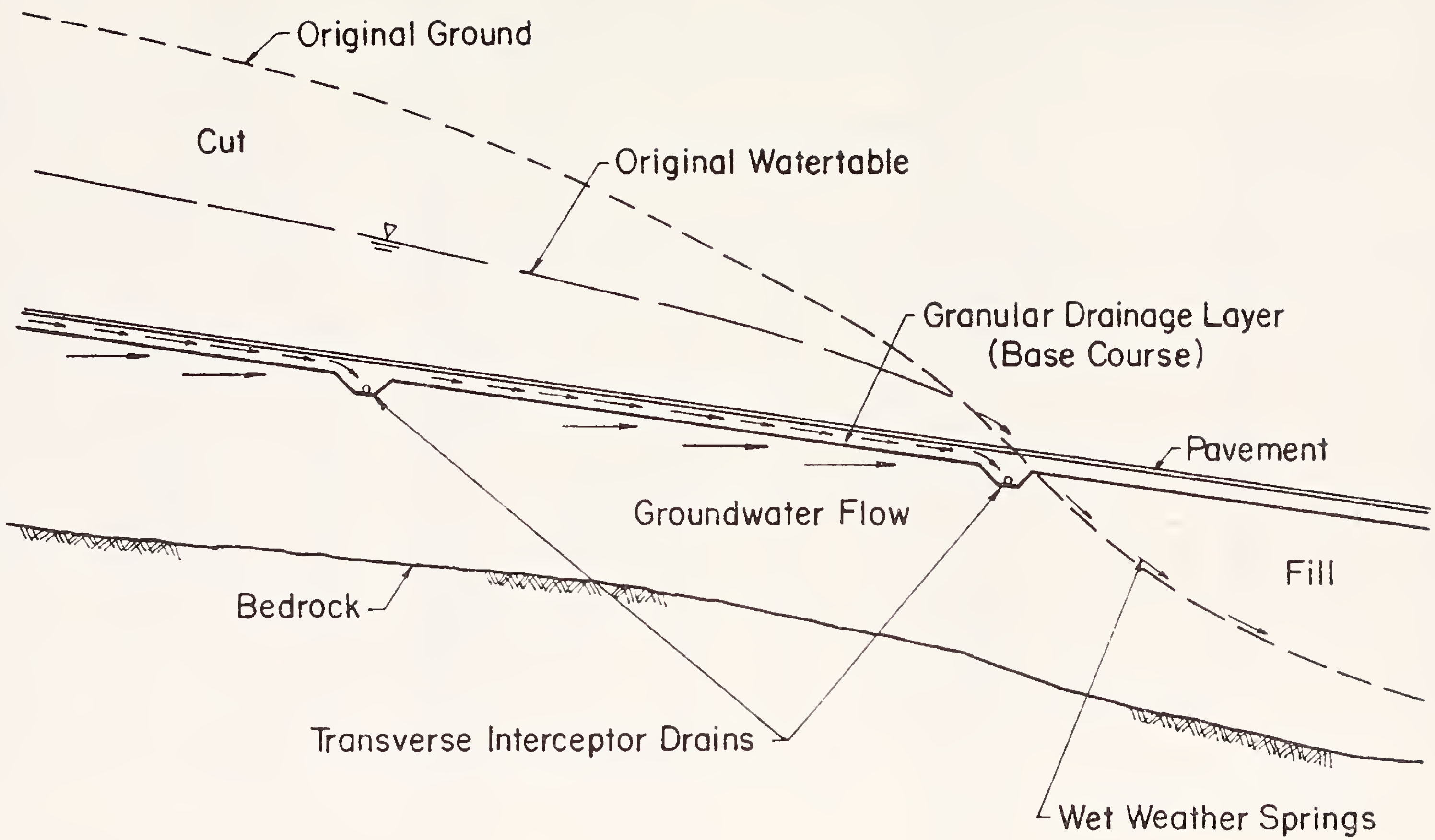


Figure 3.3. Transverse Interceptor Drain Installation In Roadway Cut With Alignment Perpendicular to Existing Contours (14).

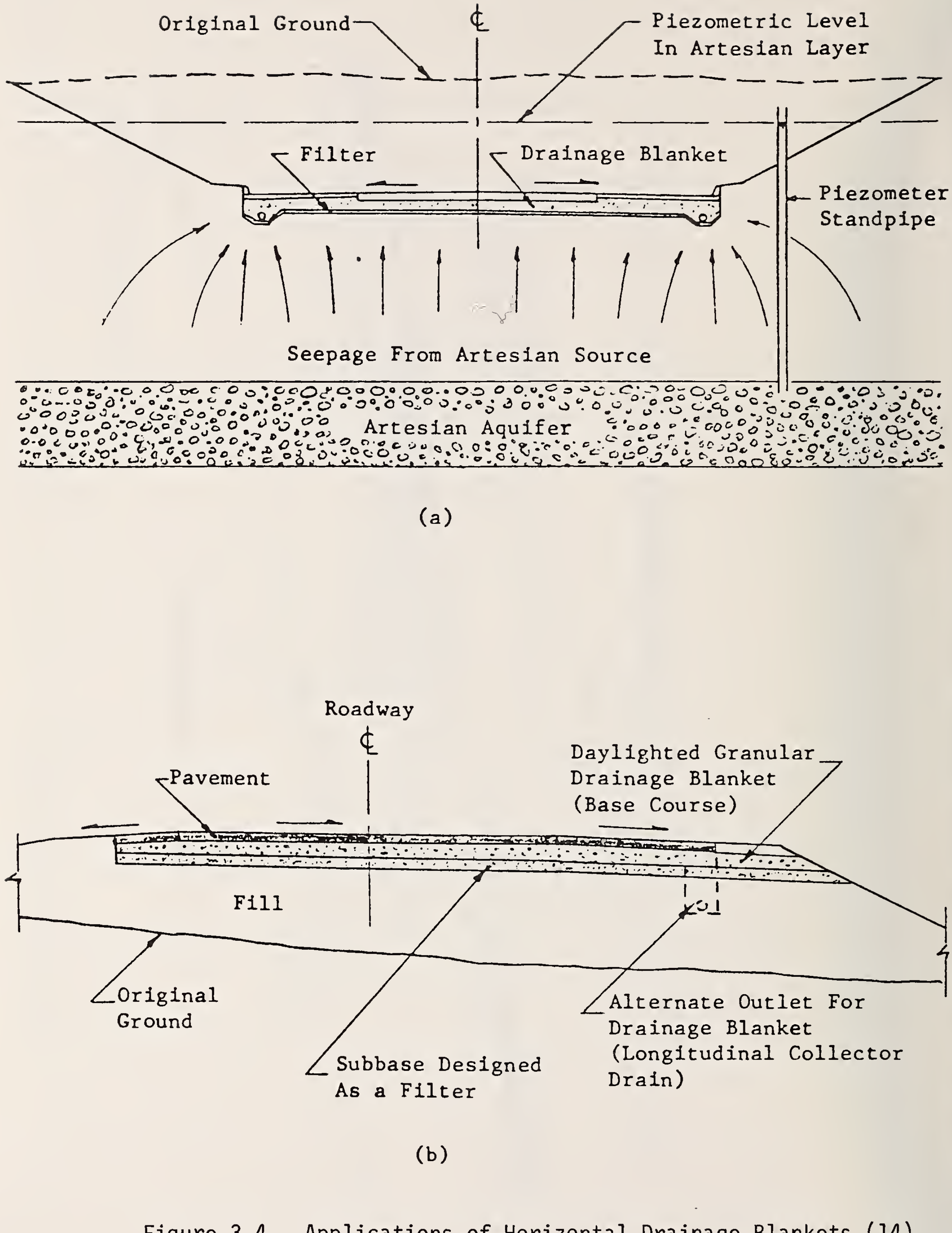


Figure 3.4. Applications of Horizontal Drainage Blankets (14).

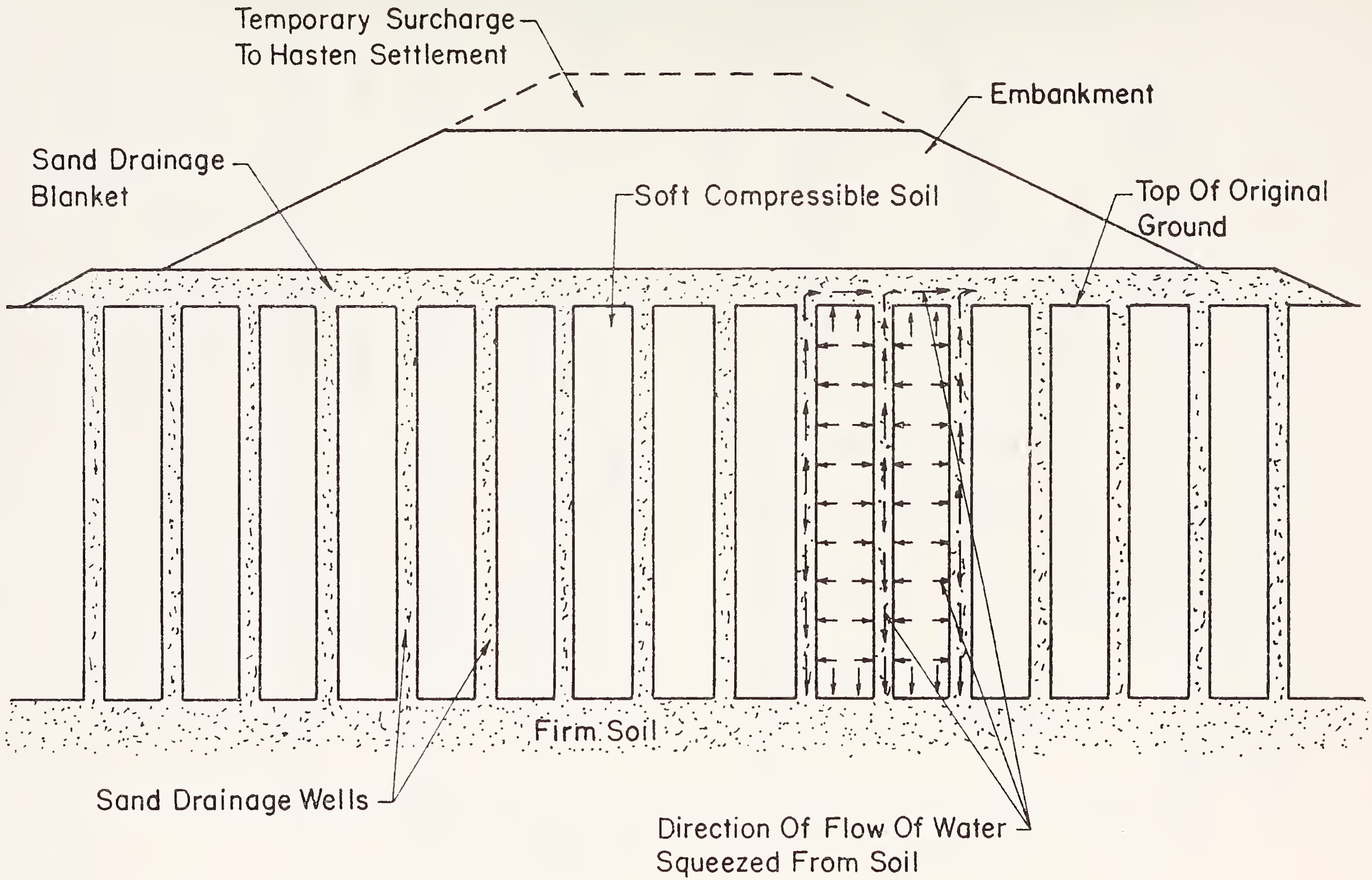


Figure 3.5. Typical Sand Drainage Well Installation (14).

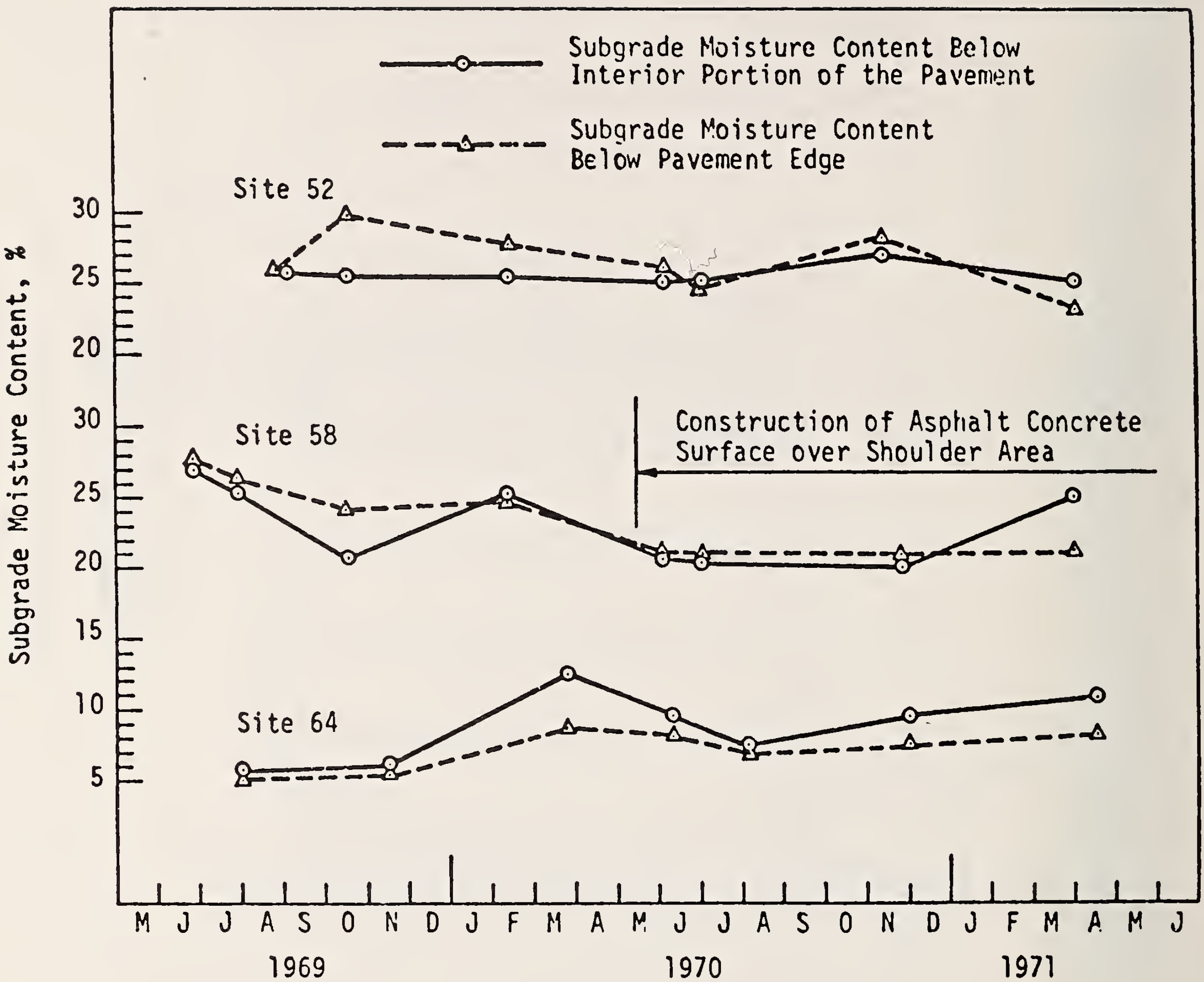


Figure 3.6. Variations in Subgrade Moisture Content with Time (24).

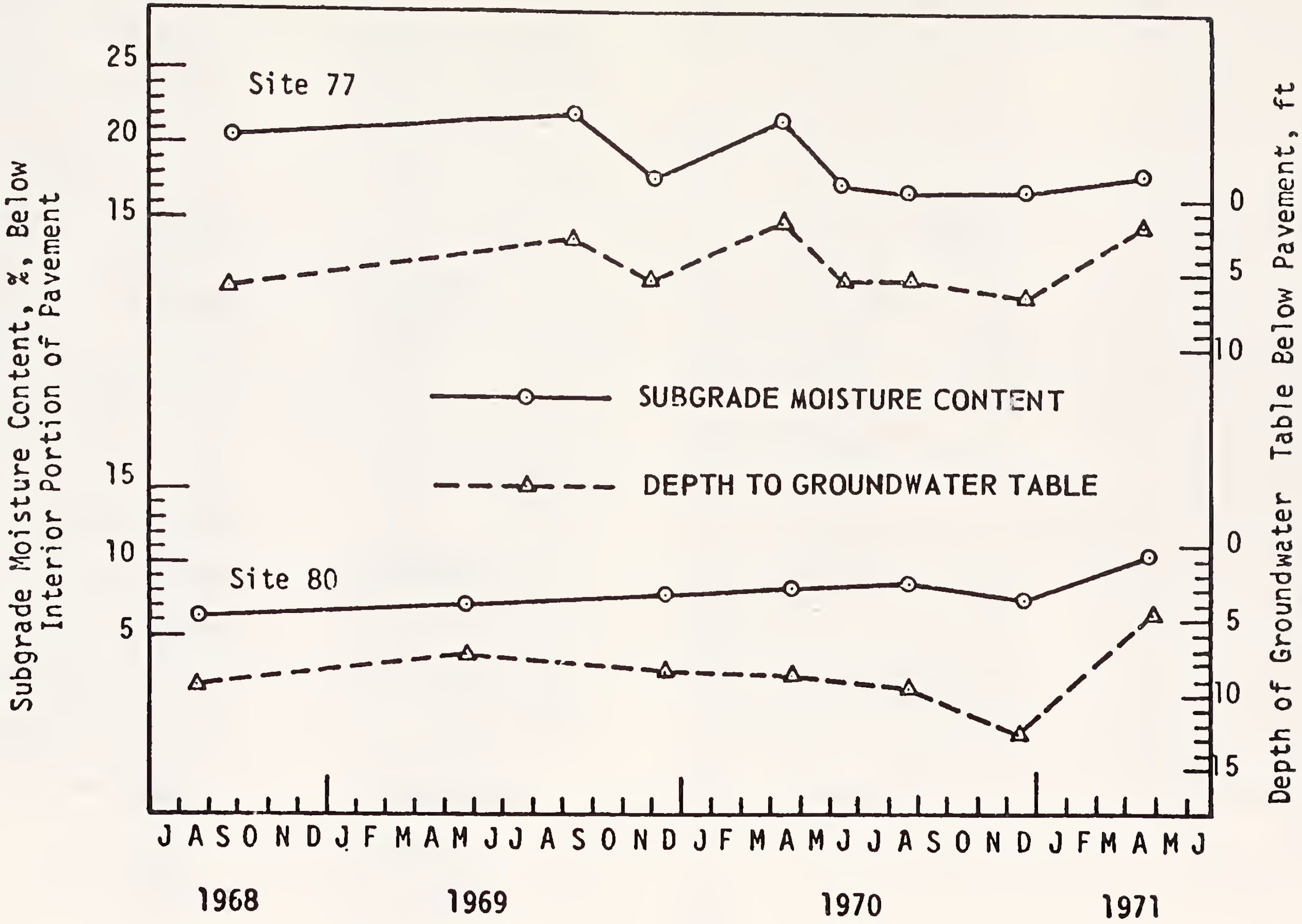
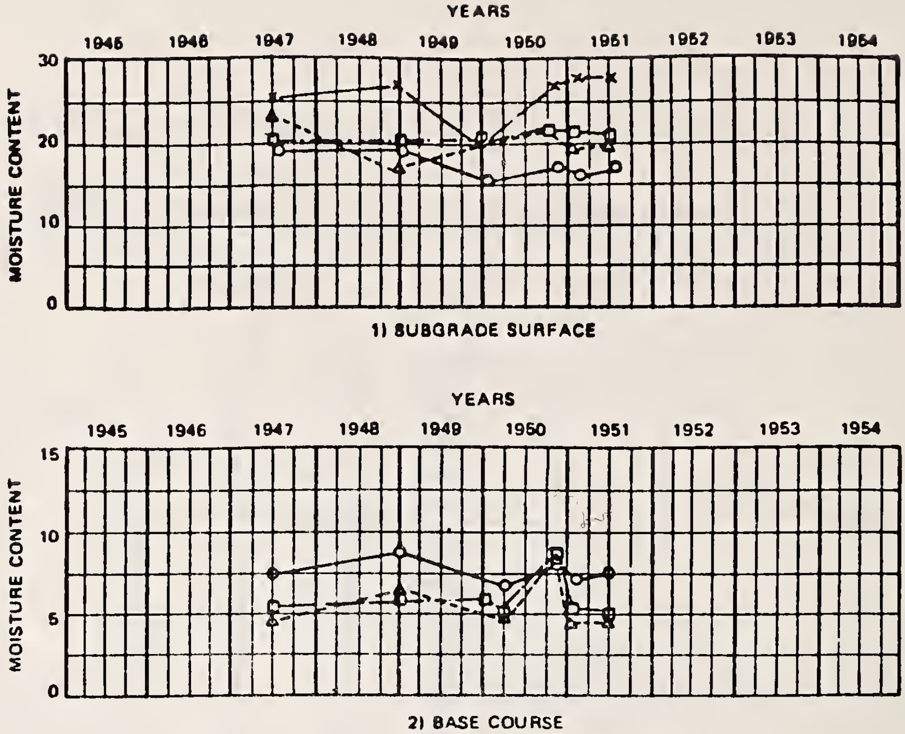
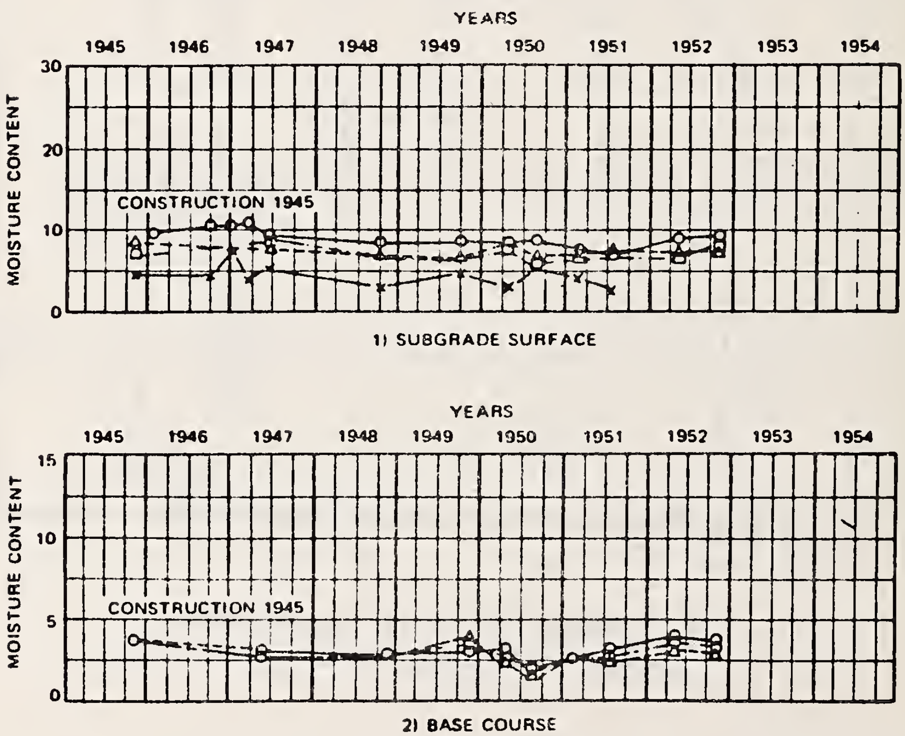


Figure 3.7. Variations in Subgrade Moisture Content Compared with Fluctuations in Groundwater Table (24).



a) Memphis Municipal Airports (35 in. of rainfall/year)



b) Kirtland Air Force Base (15 in. of rainfall/year)

Figure 3.8. Moisture Content Variations for Airfield Pavements (26).

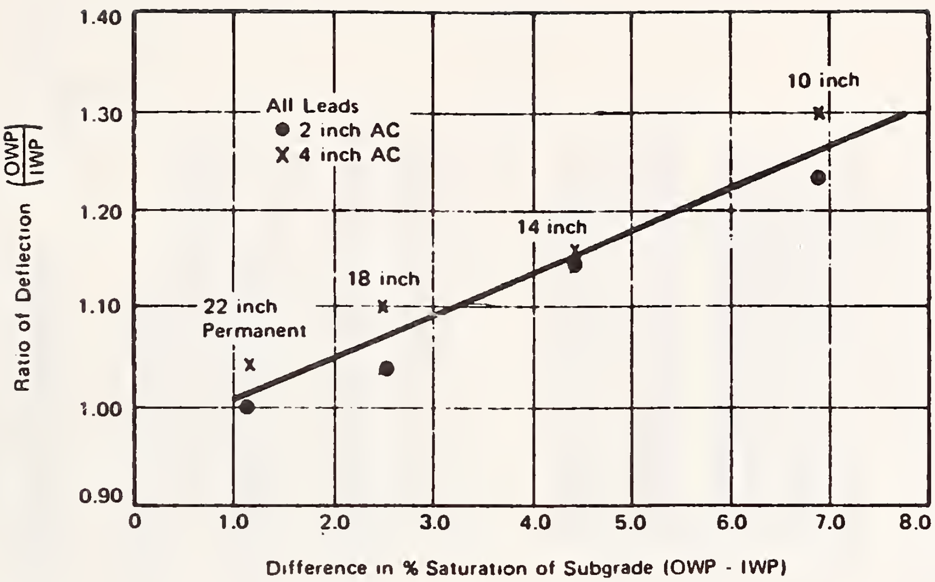


Figure 3.9. Effect of Subgrade Saturation on Pavement Deflection for the Four Total Thicknesses of Flexible Pavements - AASHO Road Test (35).

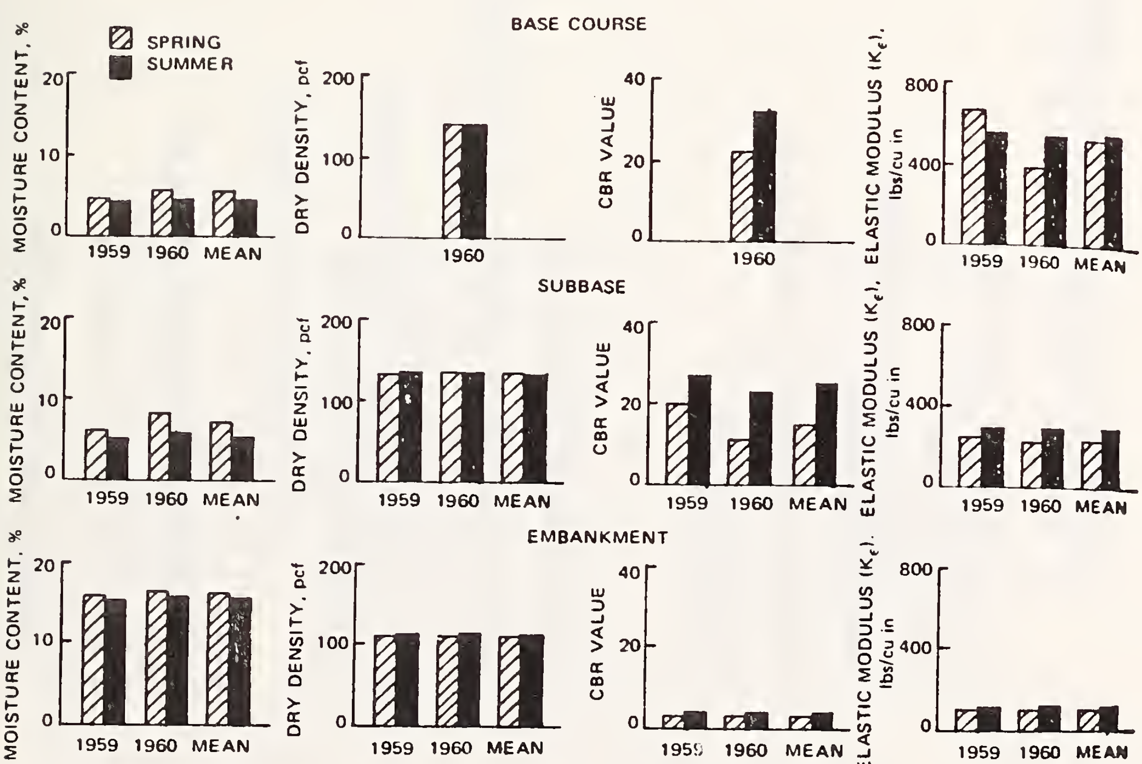


Figure 3.10. AASHO Road Test - Spring and Summer Subsurface Conditions - Loop 1, no traffic (36).

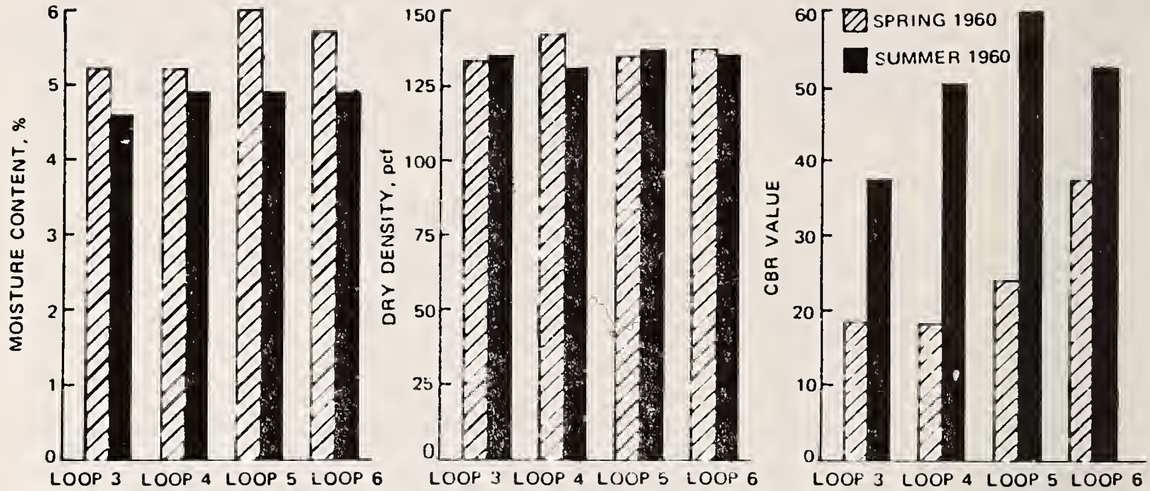


Figure 3.11. AASHO Road Test - Subbase Condition Data, Spring and Summer, 1960 (36).

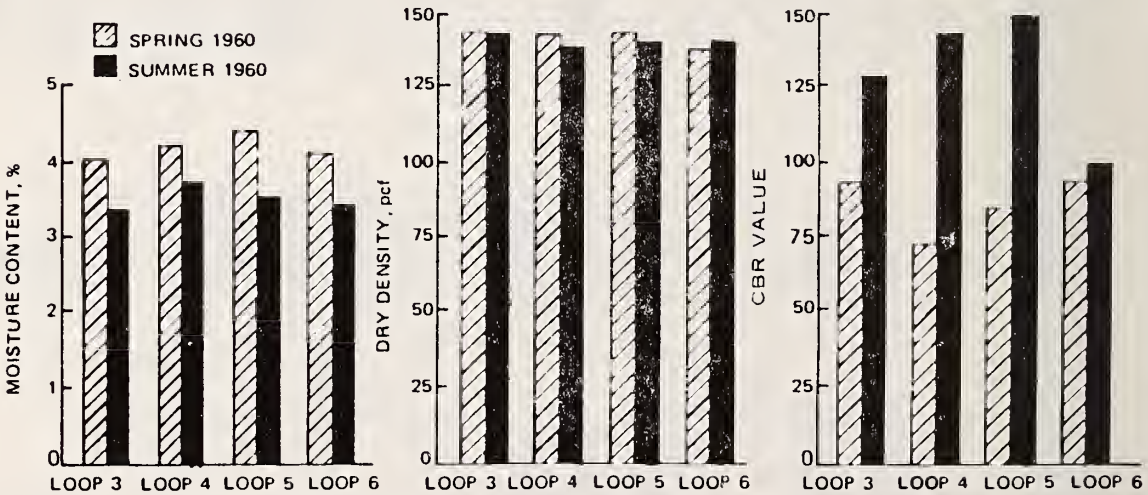


Figure 3.12. AASHO Road Test - Base Course Conditions, Spring and Summer, 1960 (36).

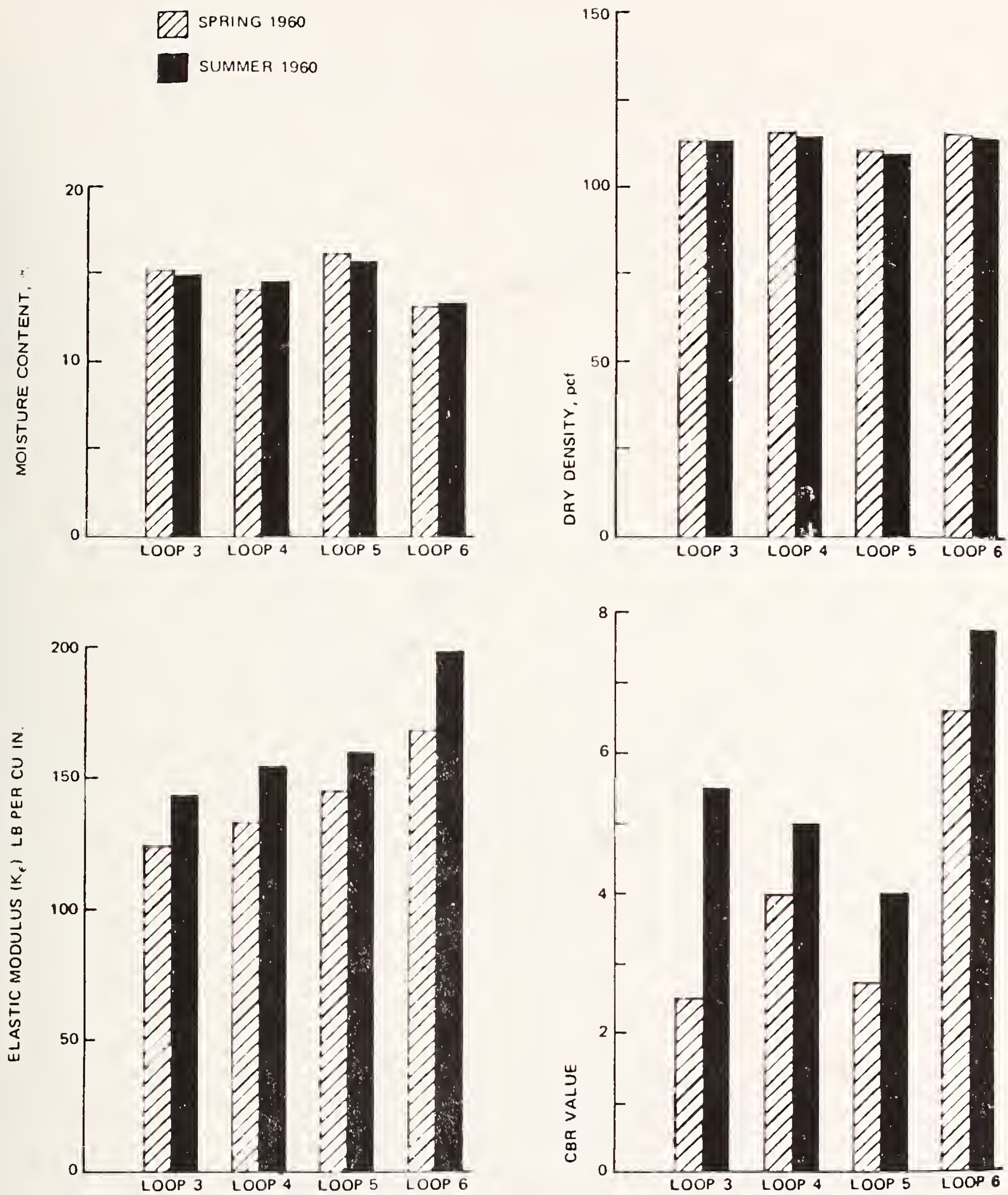


Figure 3.13. AASHO Road Test - Embankment Condition Data, Spring and Summer, 1960 (36).

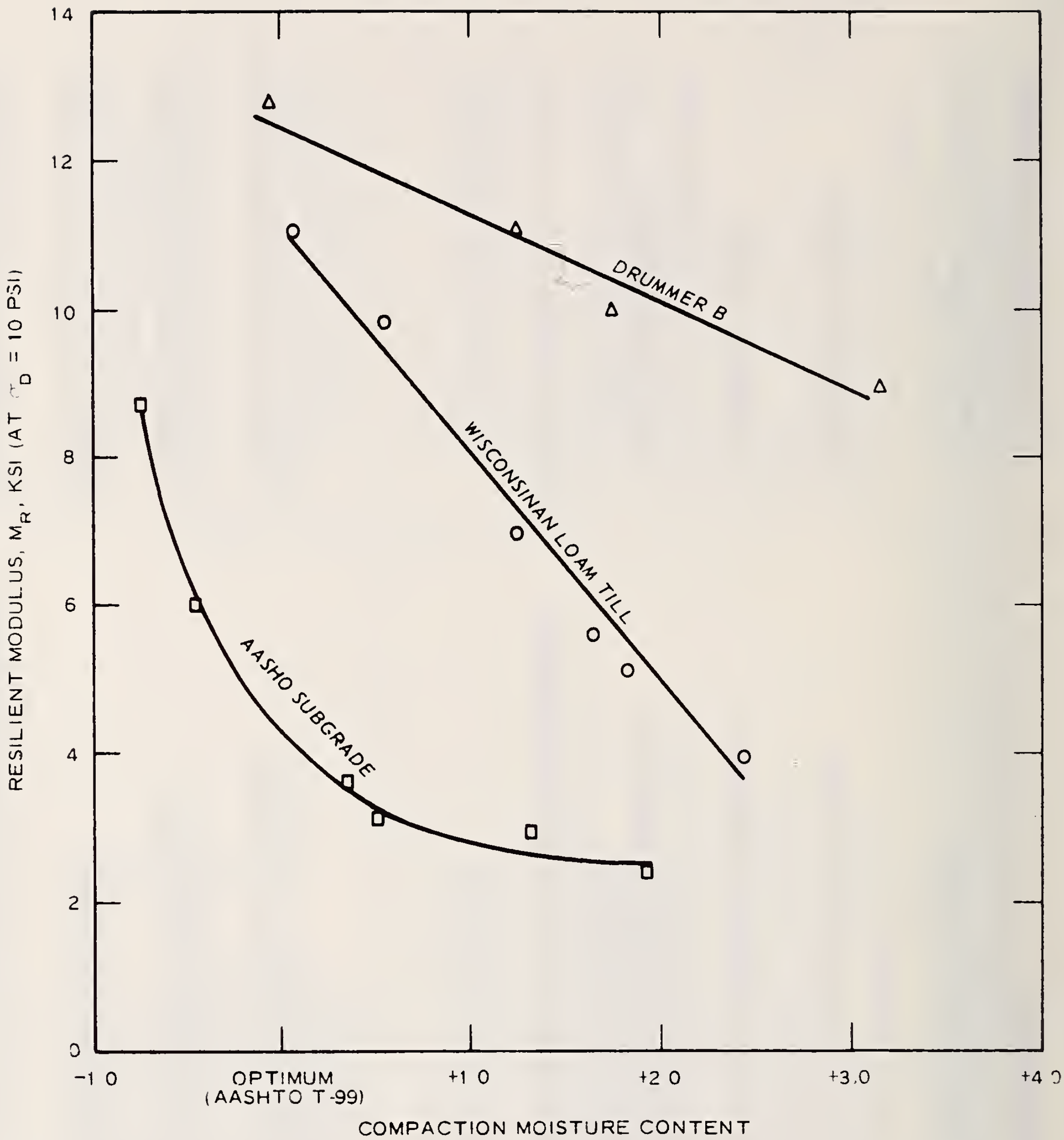


Figure 3.14. Effect of Compaction Moisture Content on Resilient Modulus (37).

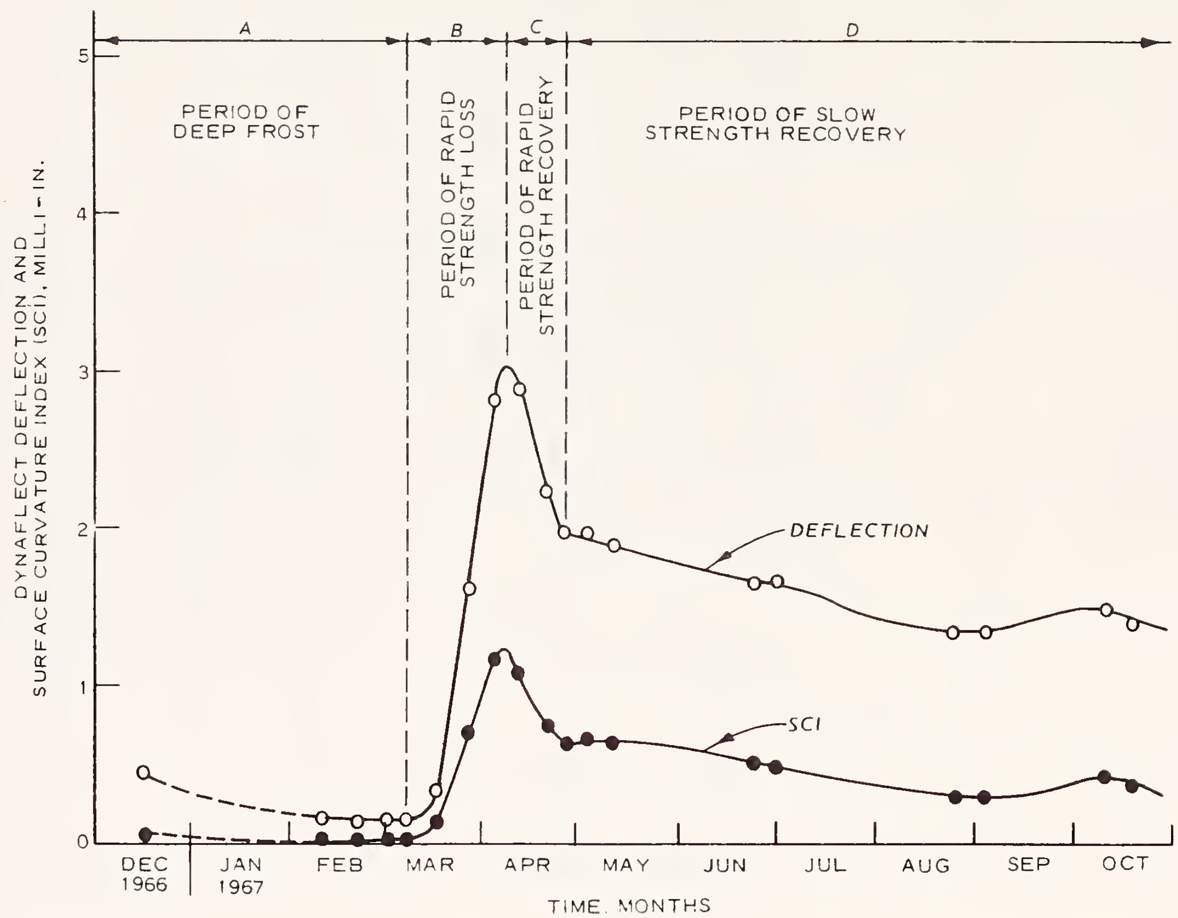


Figure 3.15. Typical Seasonal Variations in Deflections and Curvature (45).

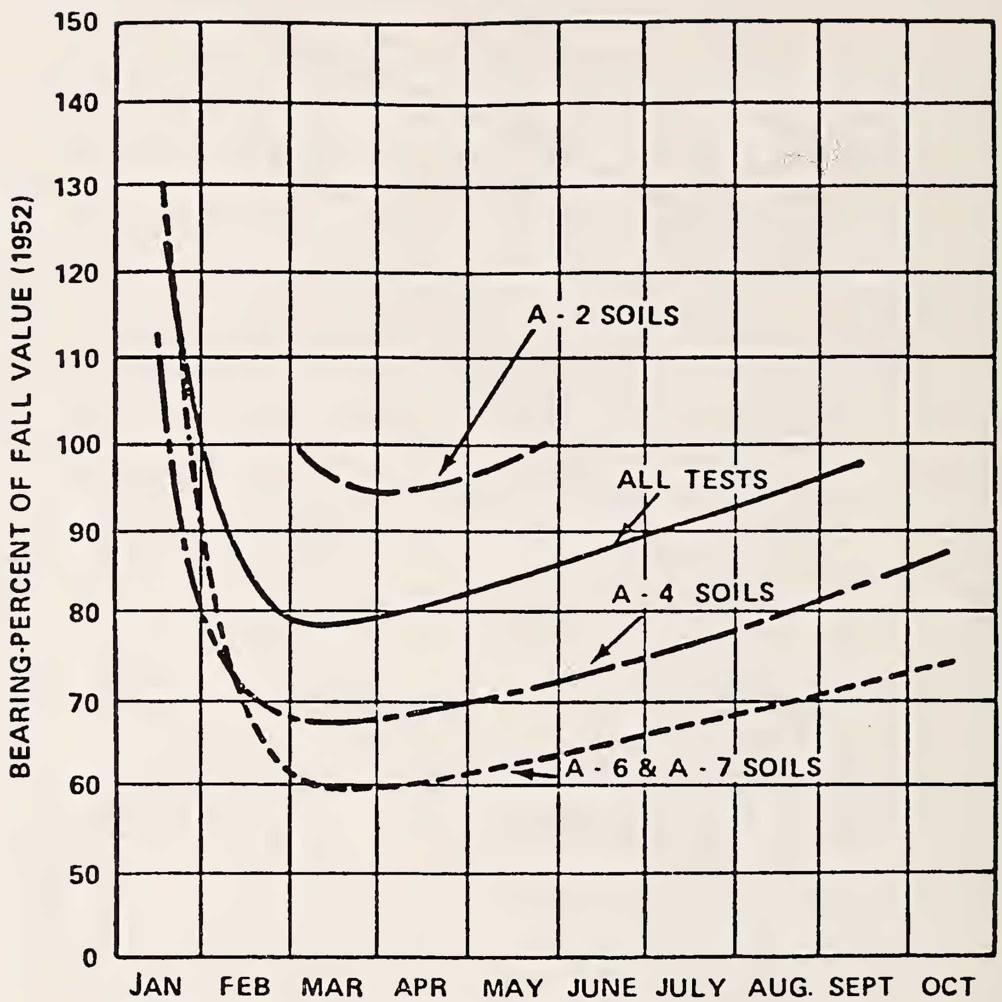


Figure 3.16. Seasonal Changes in Bearing Capacity (51).

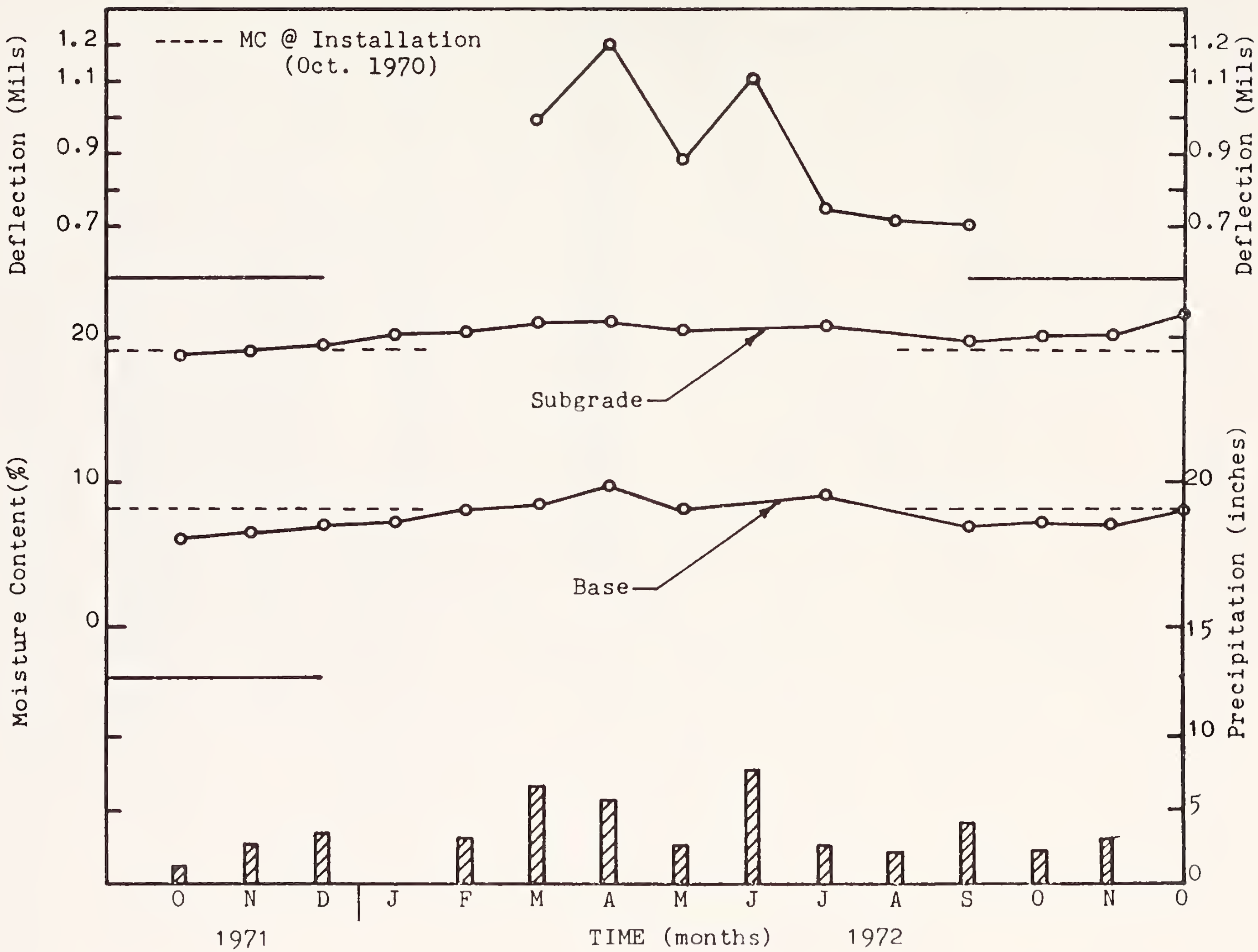


Figure 3.17. Seasonal Relationship Between Moisture and Deflection (54).

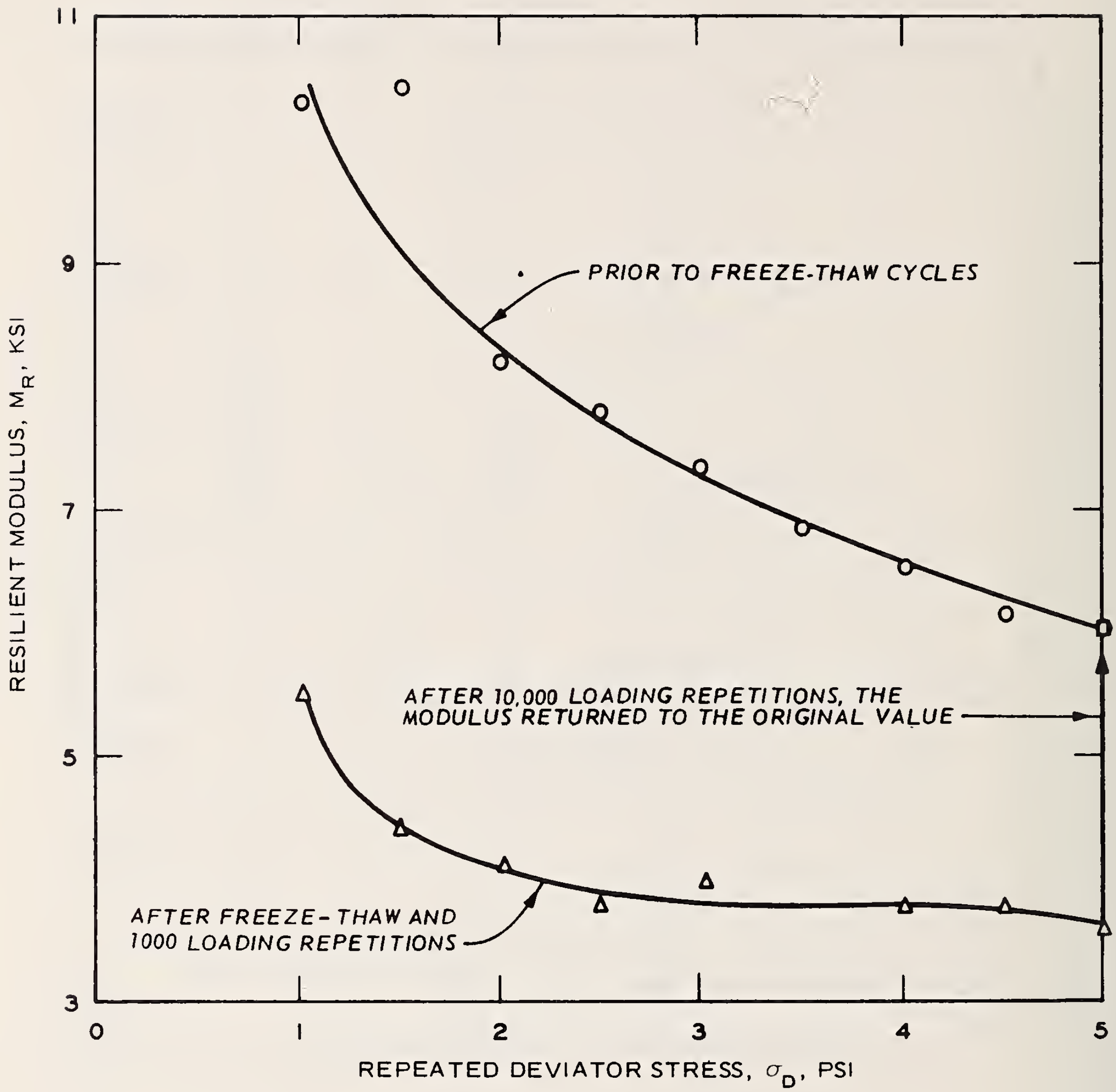


Figure 3.18. Resilient Modulus Test Results Before and After Freeze-Thaw for Undisturbed Regina Clay (58).

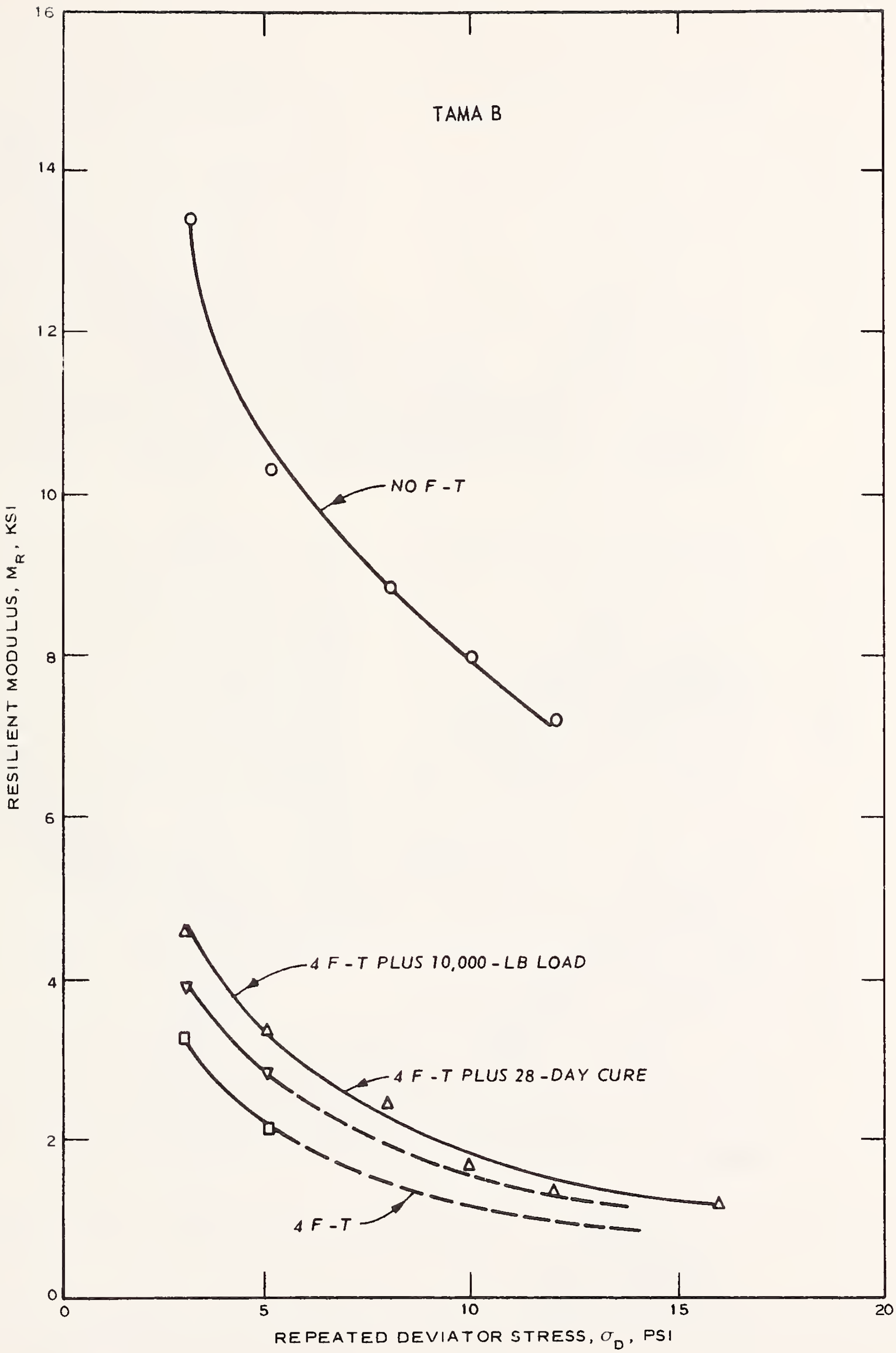


Figure 3.19. Effect of Freeze-Thaw, Additional Loading, and Additional Curing on Resilient Response of a Natural Tama B Soil (37).

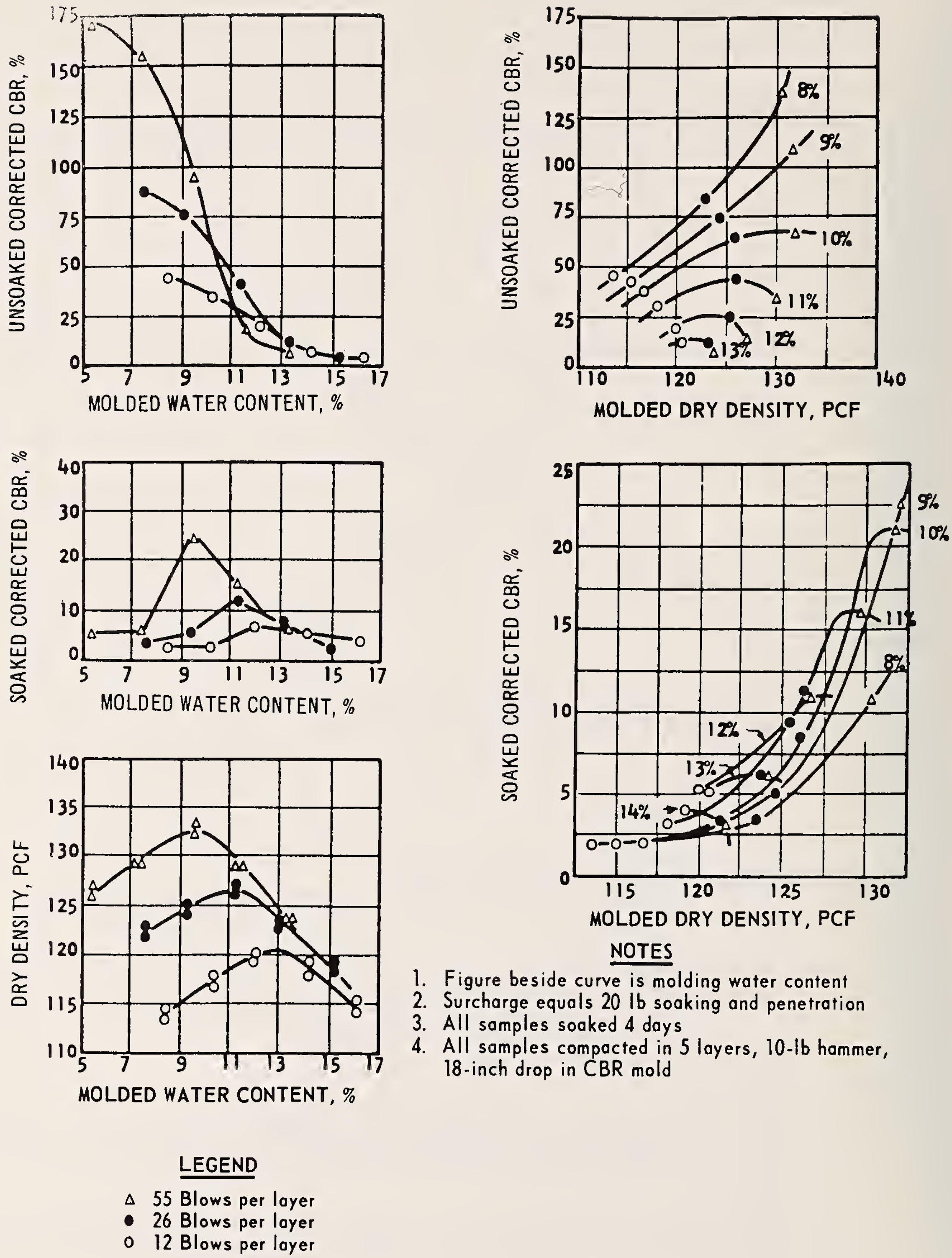


Figure 3.20. Moisture and Density Effects for the AASHO Road Test Embankment Soil (59).

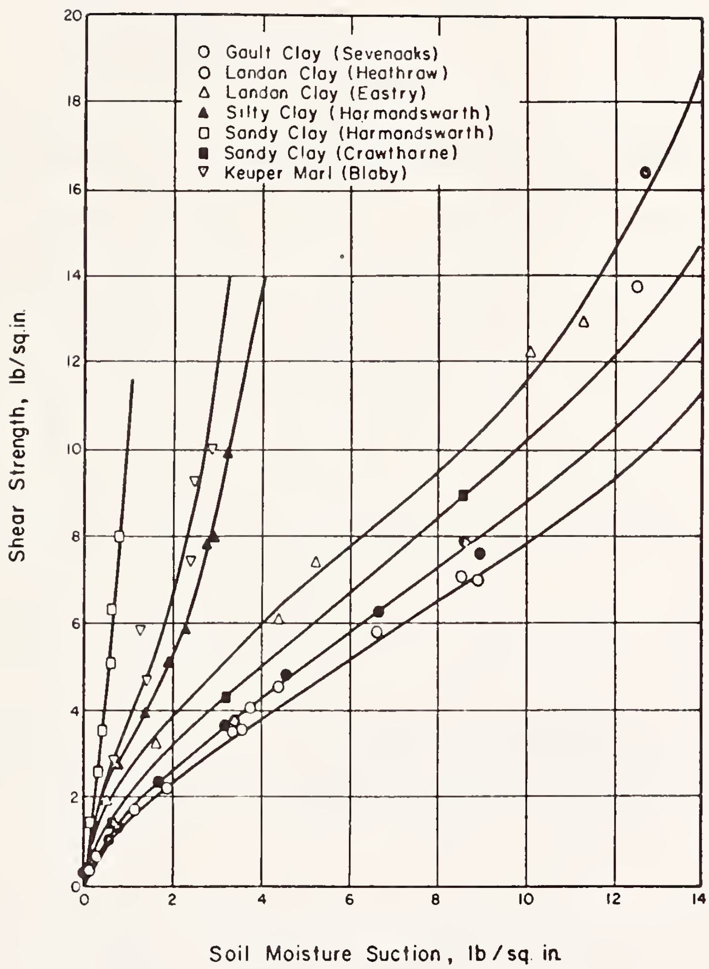


Figure 3.21. Influence of Soil Suction on Shear Strength (60).

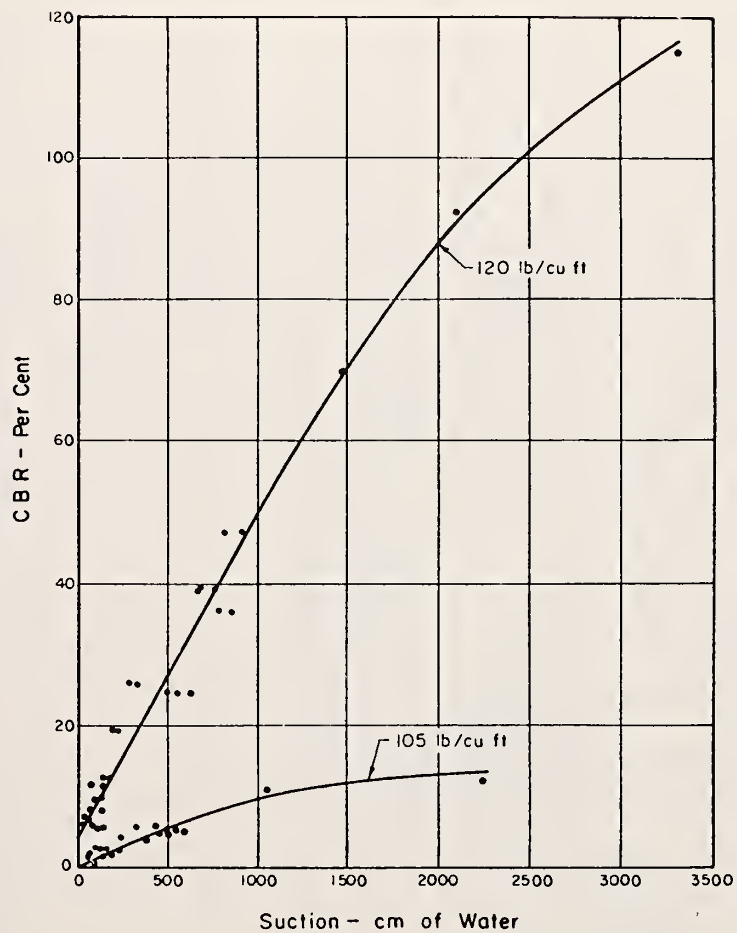


Figure 3.22. Effect of Suction on the CBR of a Silty Sand (60).

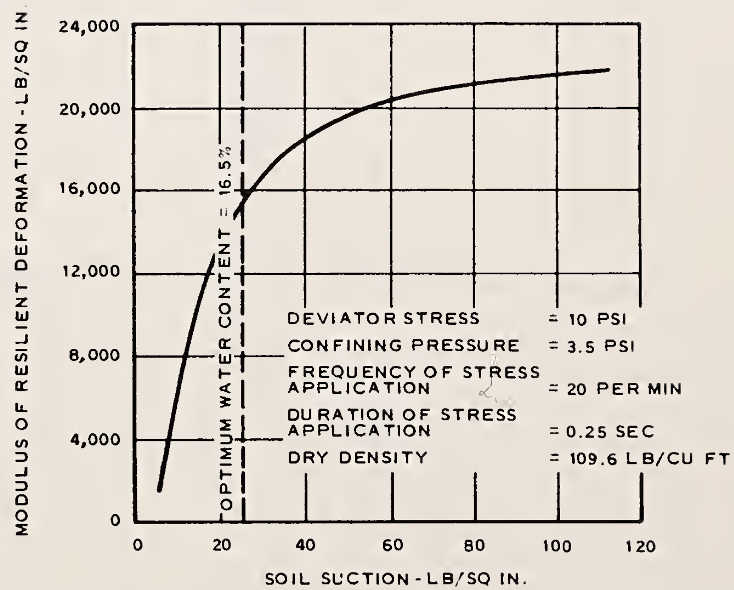


Figure 3.23. Effect of Soil Suction on the Resilient Modulus of a Canadian Till (61).

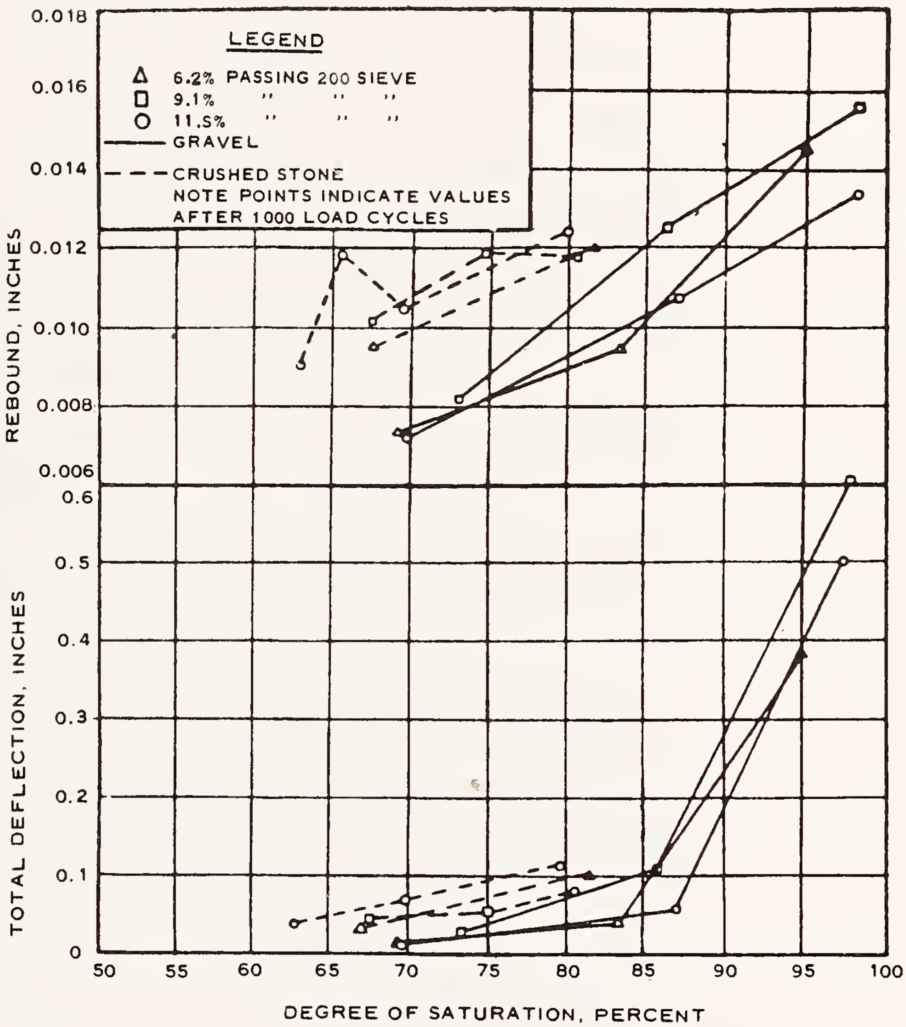


Figure 3.24. Effect of Degree of Saturation on the Repeated-Load Deformation Properties of the AASHO Granular Materials (62).

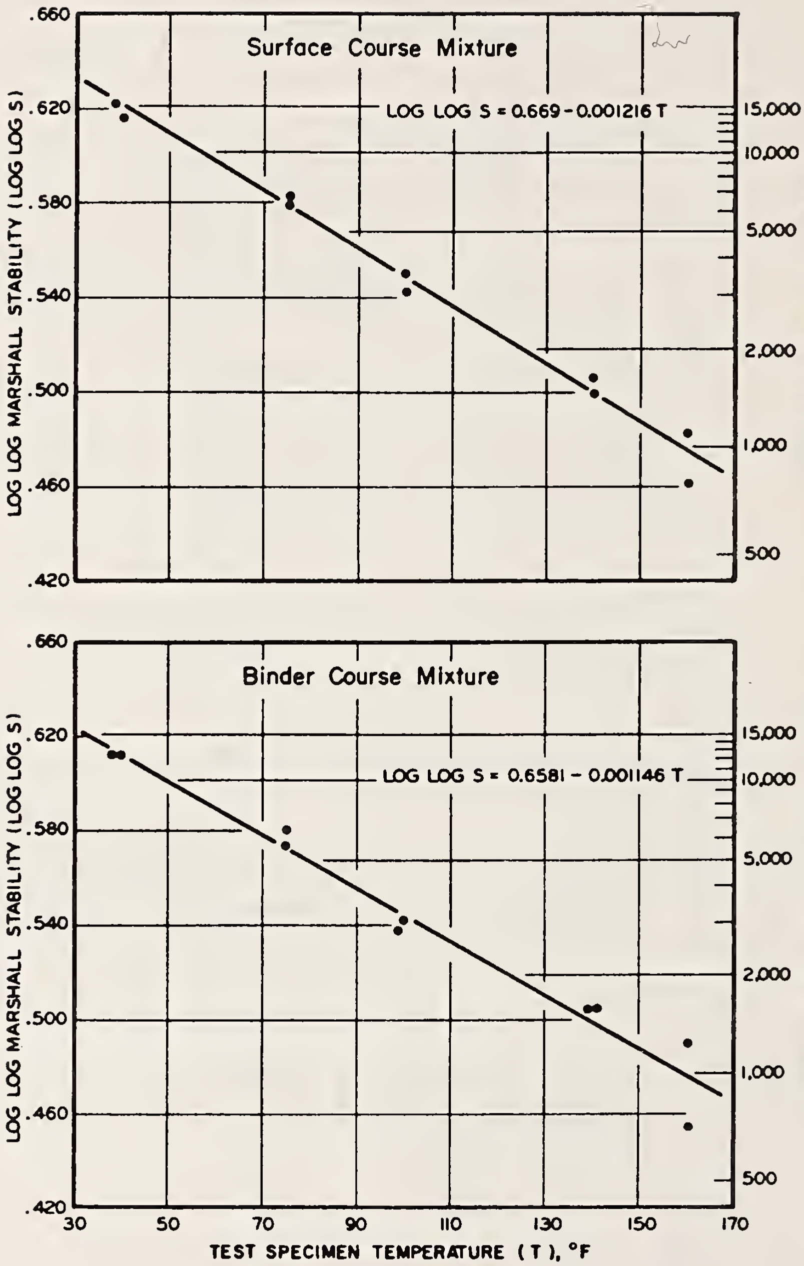


Figure 3.25. Influence of Temperature on the Marshall Stability of the AASHO Asphaltic Concrete Mixtures (36).

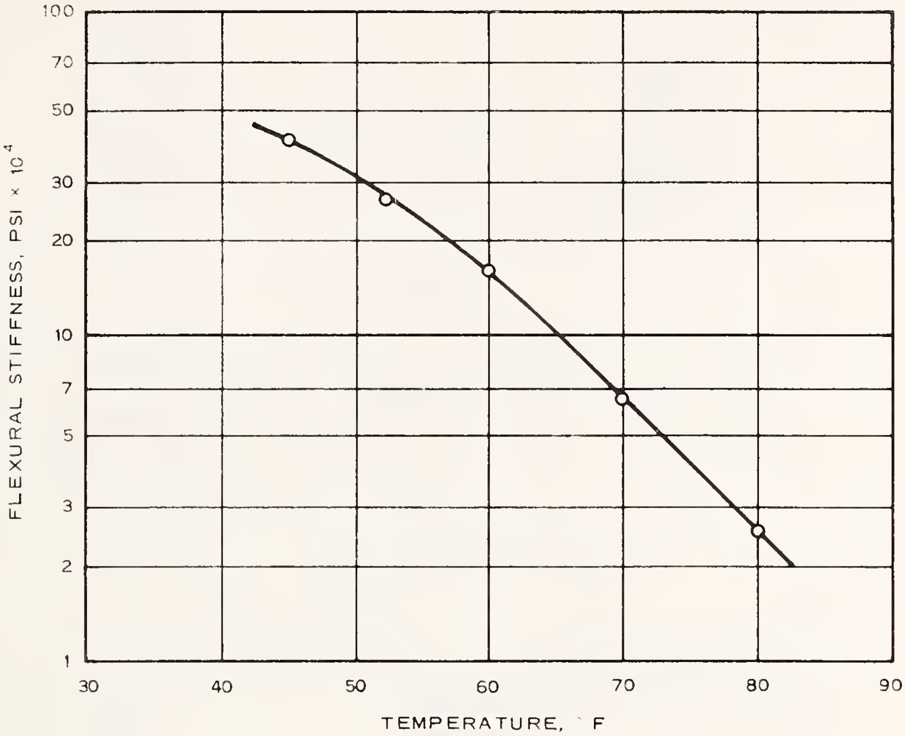


Figure 3.26. Influence of Temperature on Flexural Stiffness of an Asphaltic Concrete (65).

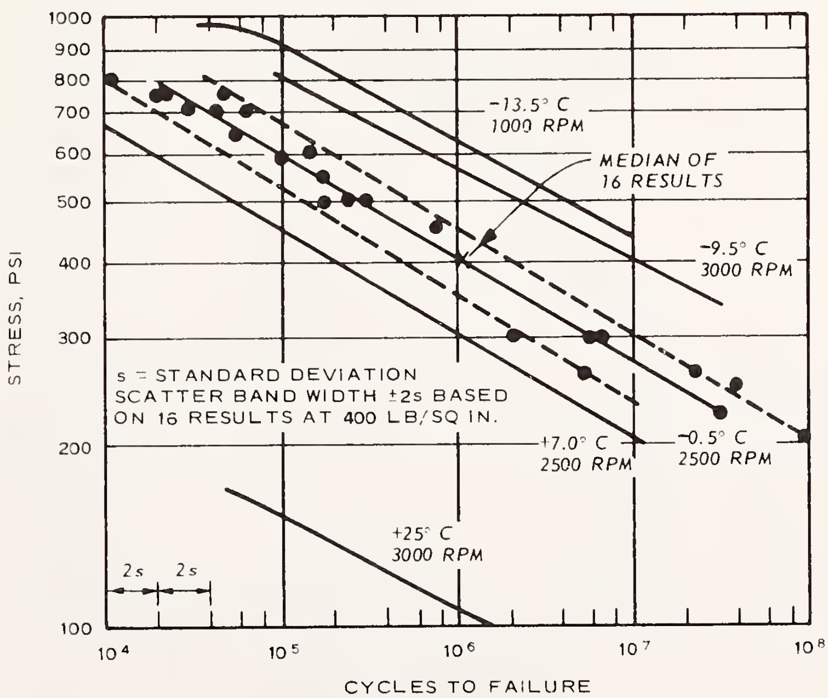


Figure 3.27. Effect of Temperature on the Fatigue Response of an Asphaltic Concrete Paving Mixture (68).

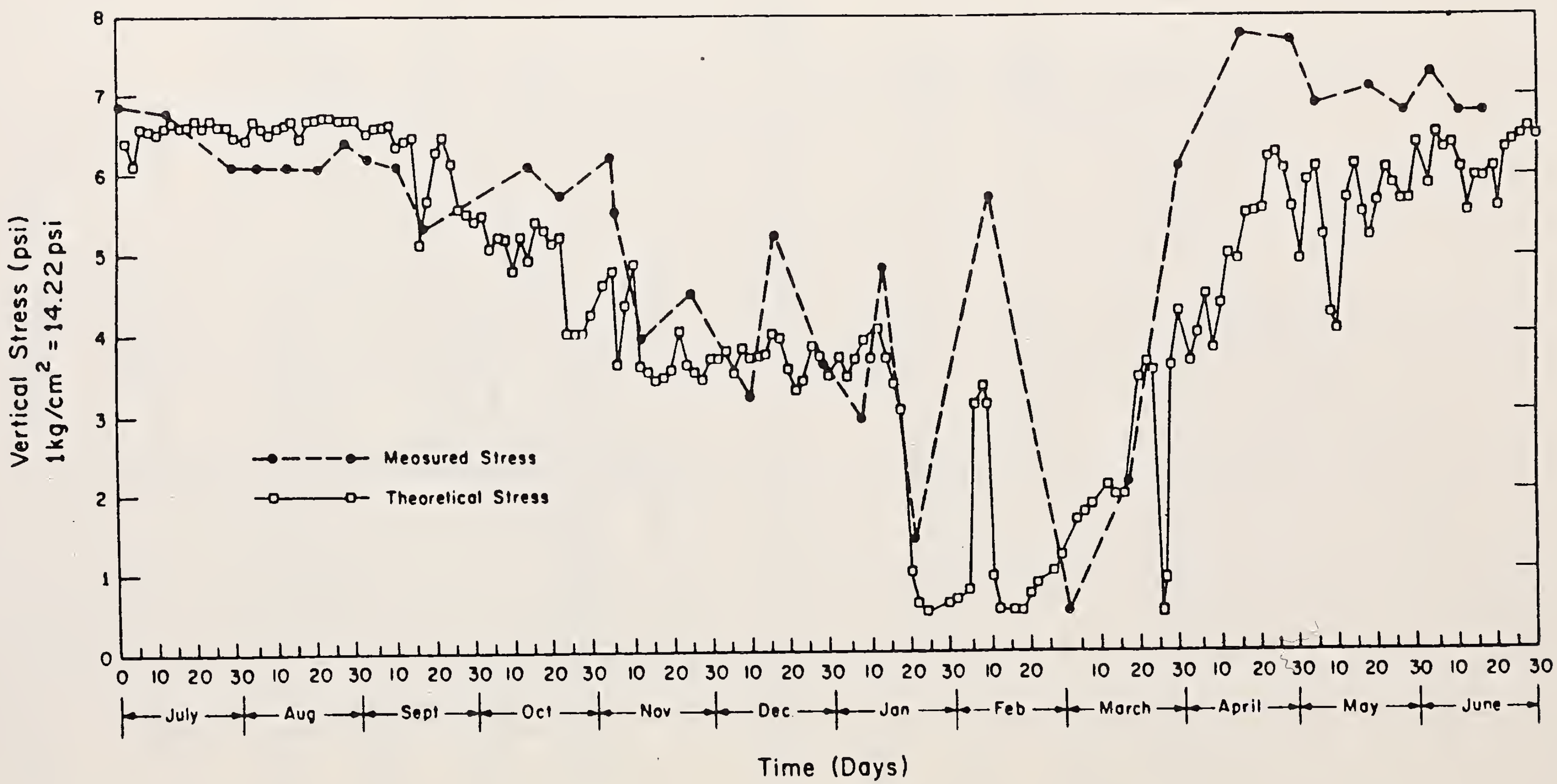


Figure 3.28. Influence of Seasonal Variations on the Theoretical and Measured Stresses at the Subbase-Subgrade Interface (AASHO Test Section 581, July, 1959, to June 30, 1960) (69).

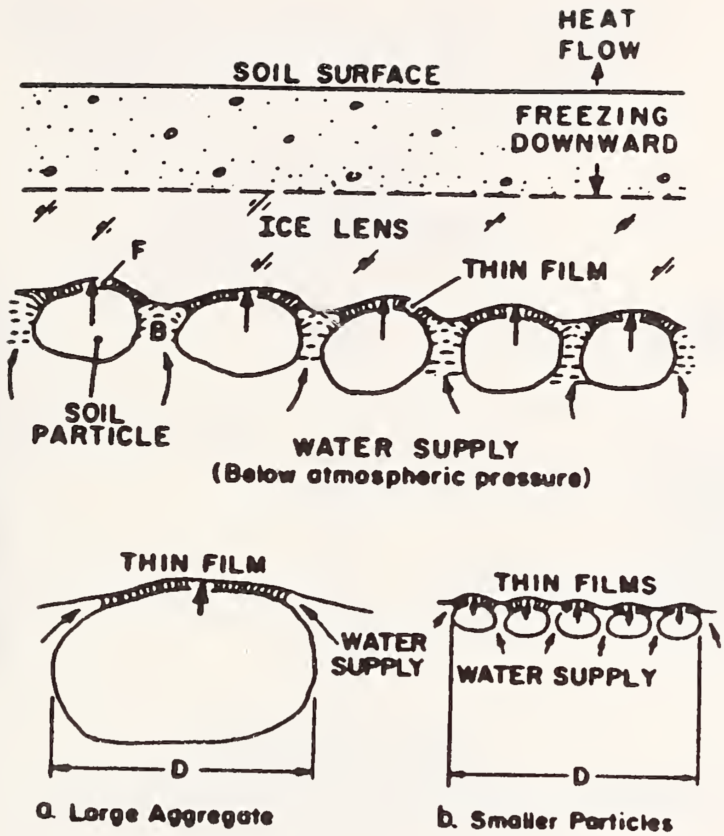


Figure 3.29. Frost Heave Mechanisms (75).

VOLUME CHANGES

	MEGASCALE →	→	→	MICROSCALE
PROCESS	UNLOAD	LOAD	HYDRATE	DESSICATE
RESPONSE	Heave Rebound	Sink Consolidate	Expand Swell	Collapse Shrink

Figure 3.30. Nature of Volume Changes in Clay Soils (80).

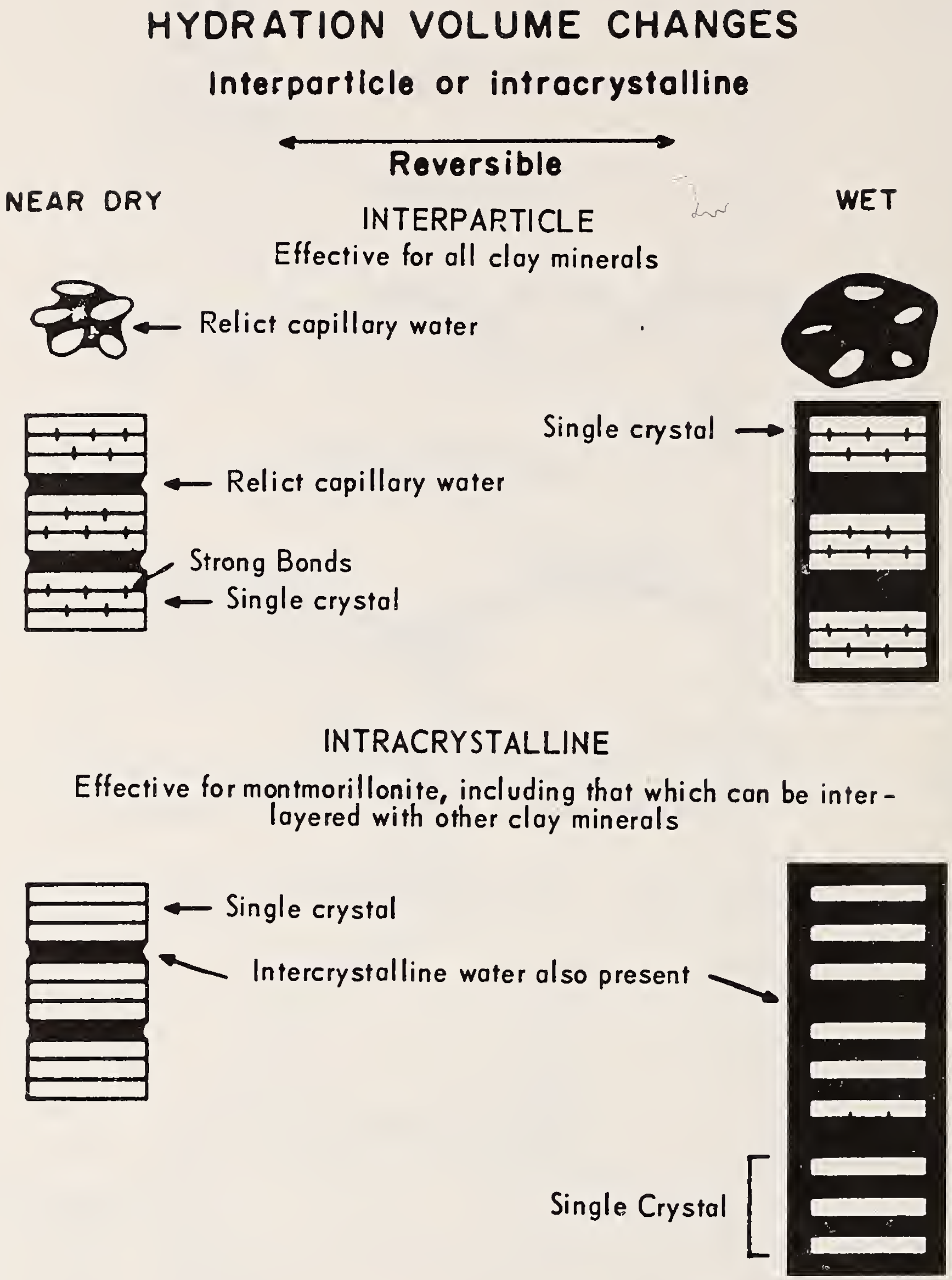


Figure 3.31. Nature of Hydration Volume Changes (80).

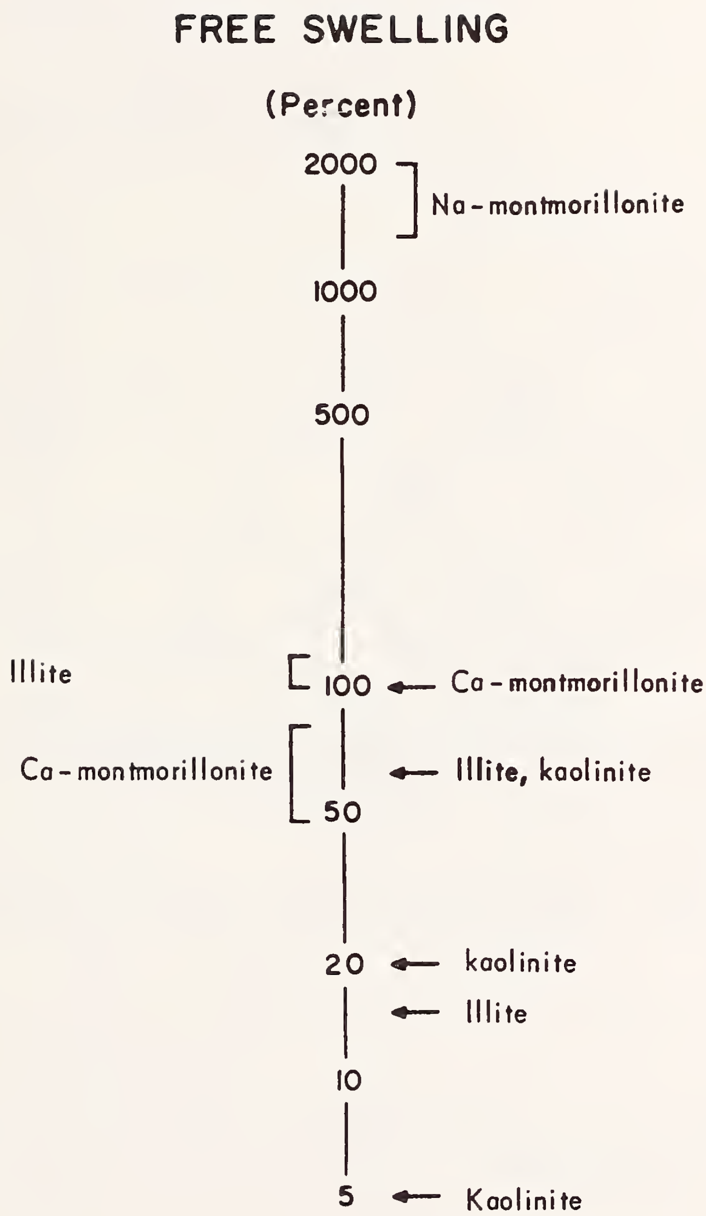


Figure 3.32. Free Swell for Clay Minerals (80).

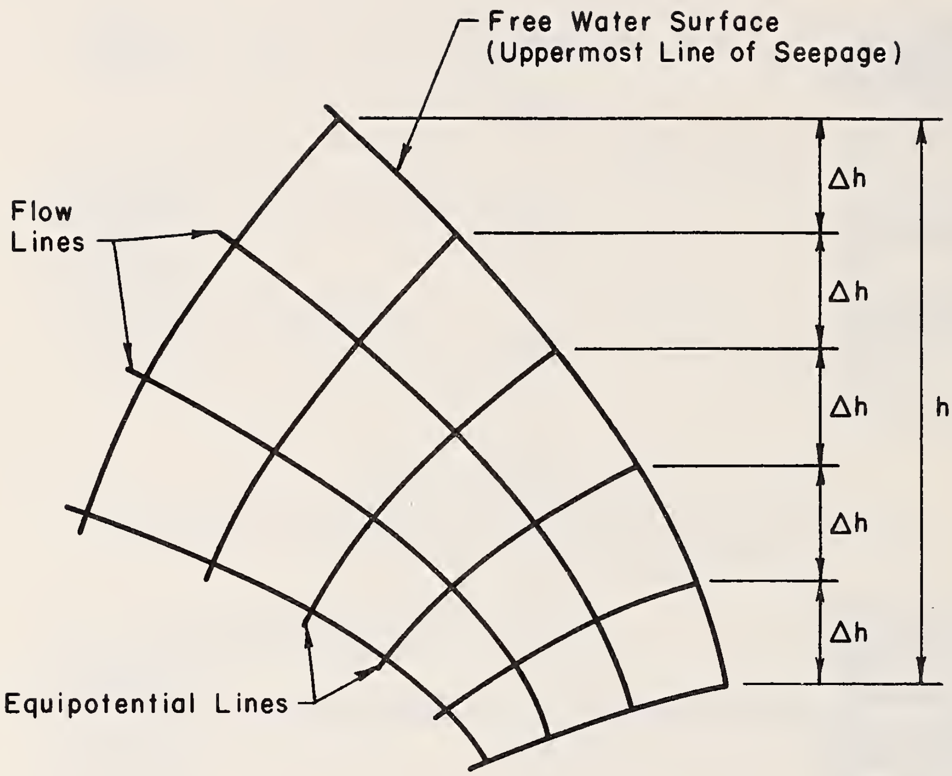
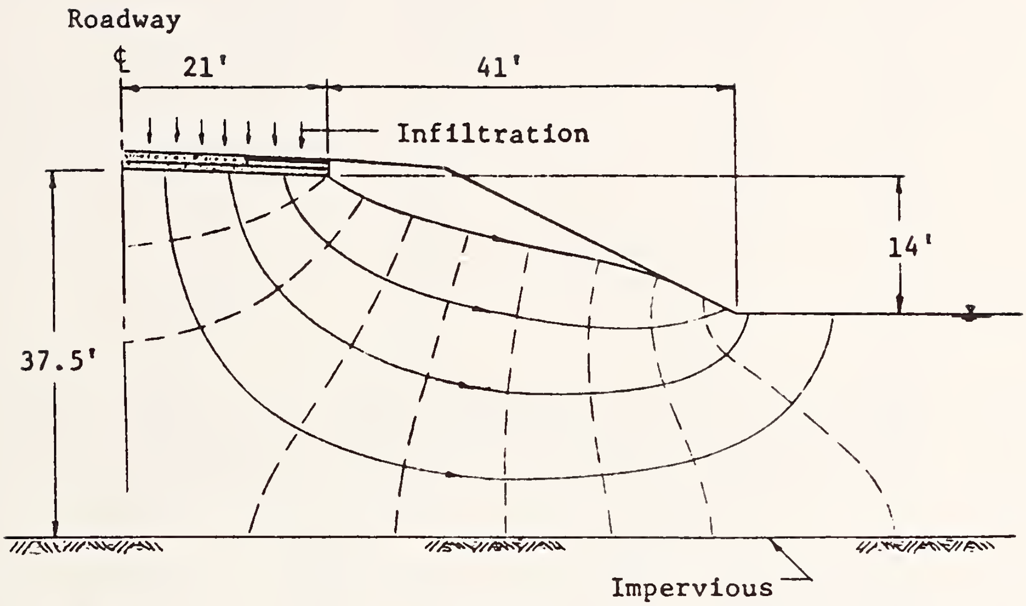
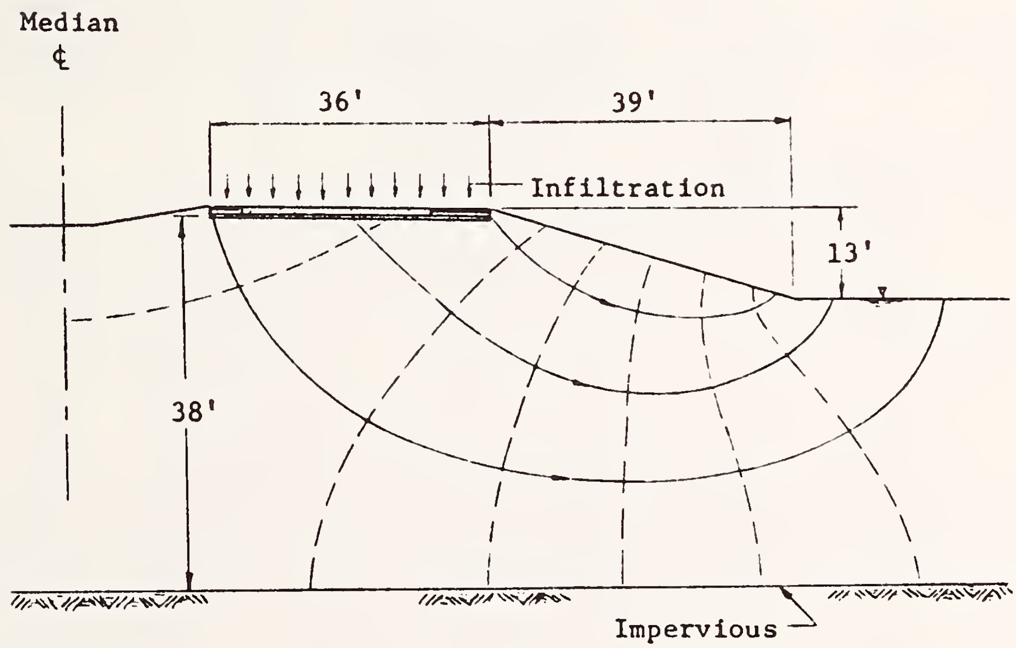


Figure 3.33. Flow Net Showing Flow Lines and Equipotential Lines (92).



(a)



(b)

Figure 3.34. Flow Nets Showing Seepage Through a Subgrade (14).

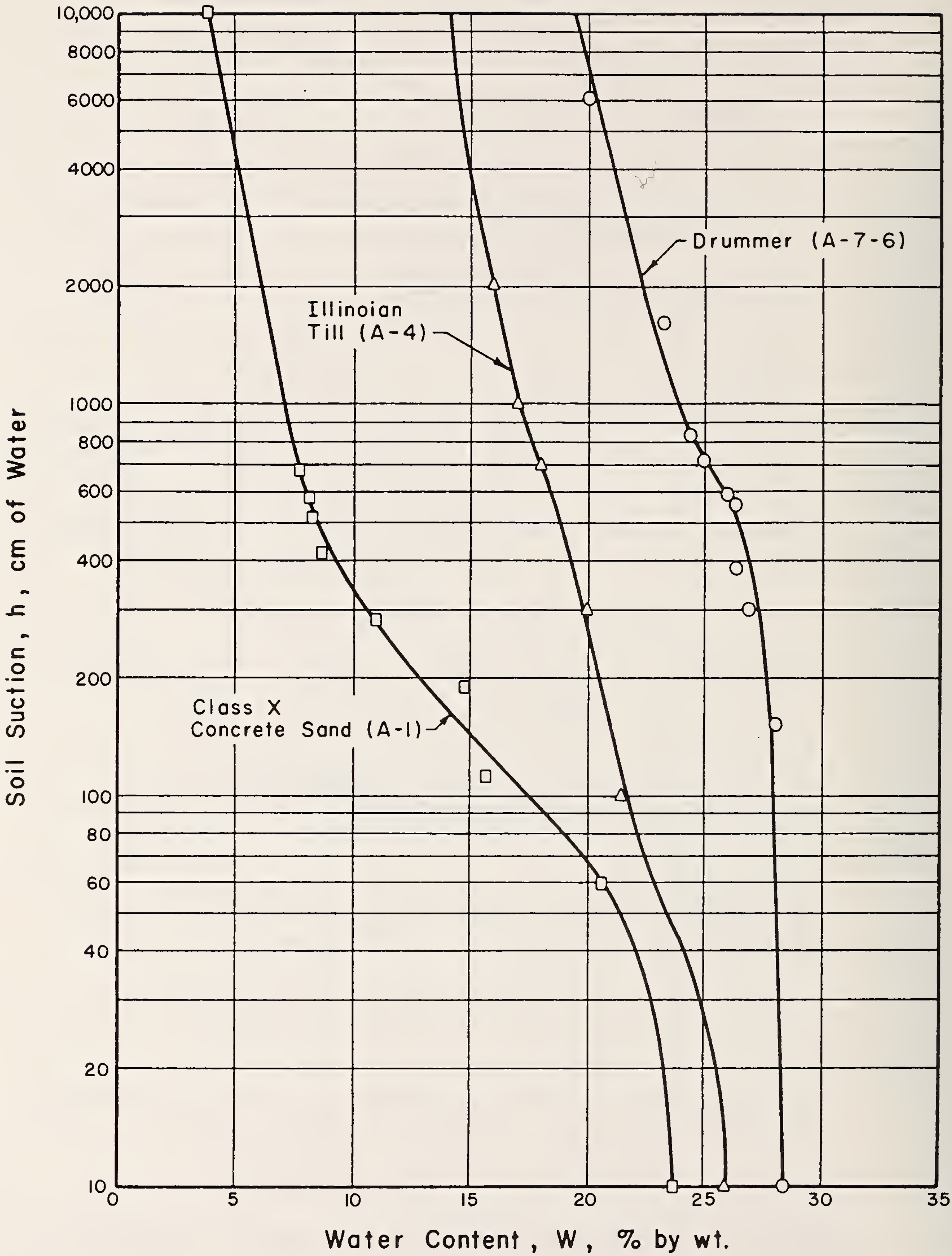


Figure 3.35. Soil Water Content-Suction Characteristic Relationship (97).

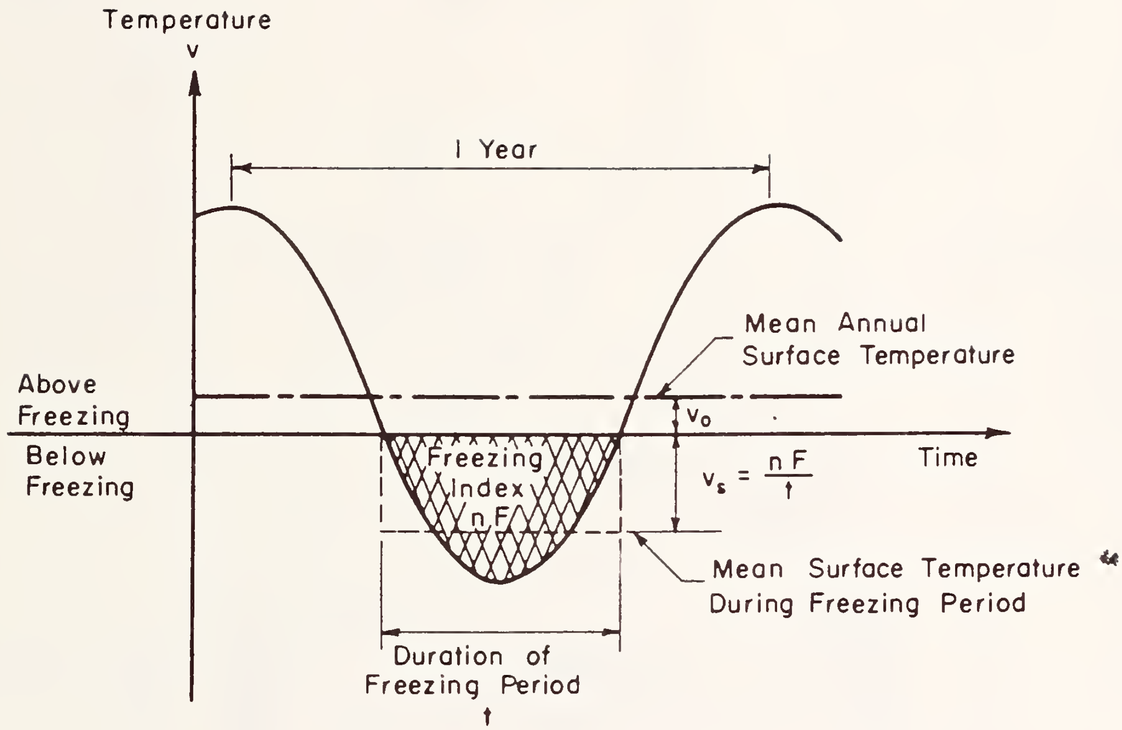


Figure 3.36. Sine Curve for an Ideal Surface Temperature Variation (114).

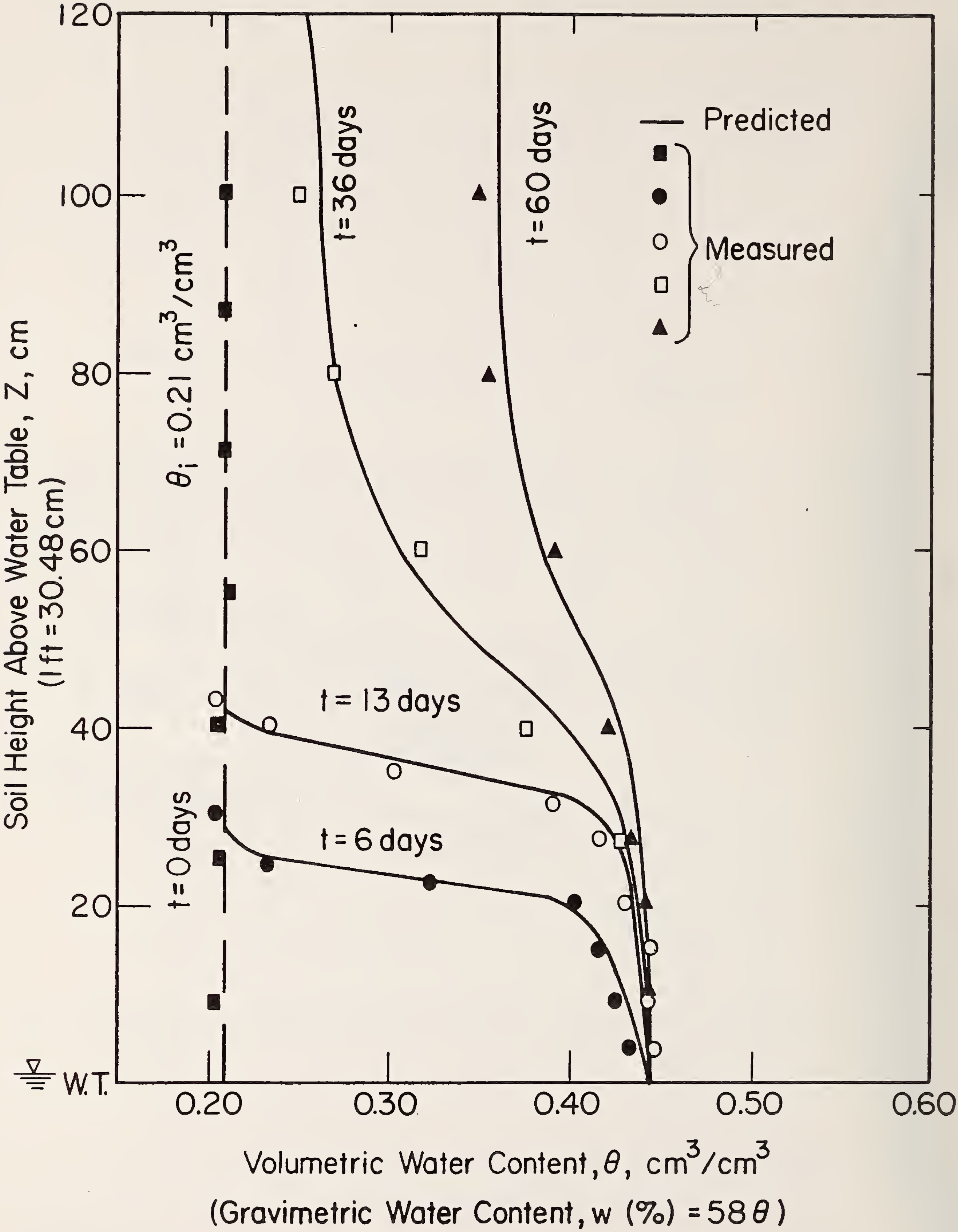


Figure 3.37. Comparison Between Predicted and Measured Moisture Contents for Illinoian Till above a Water Table (10).

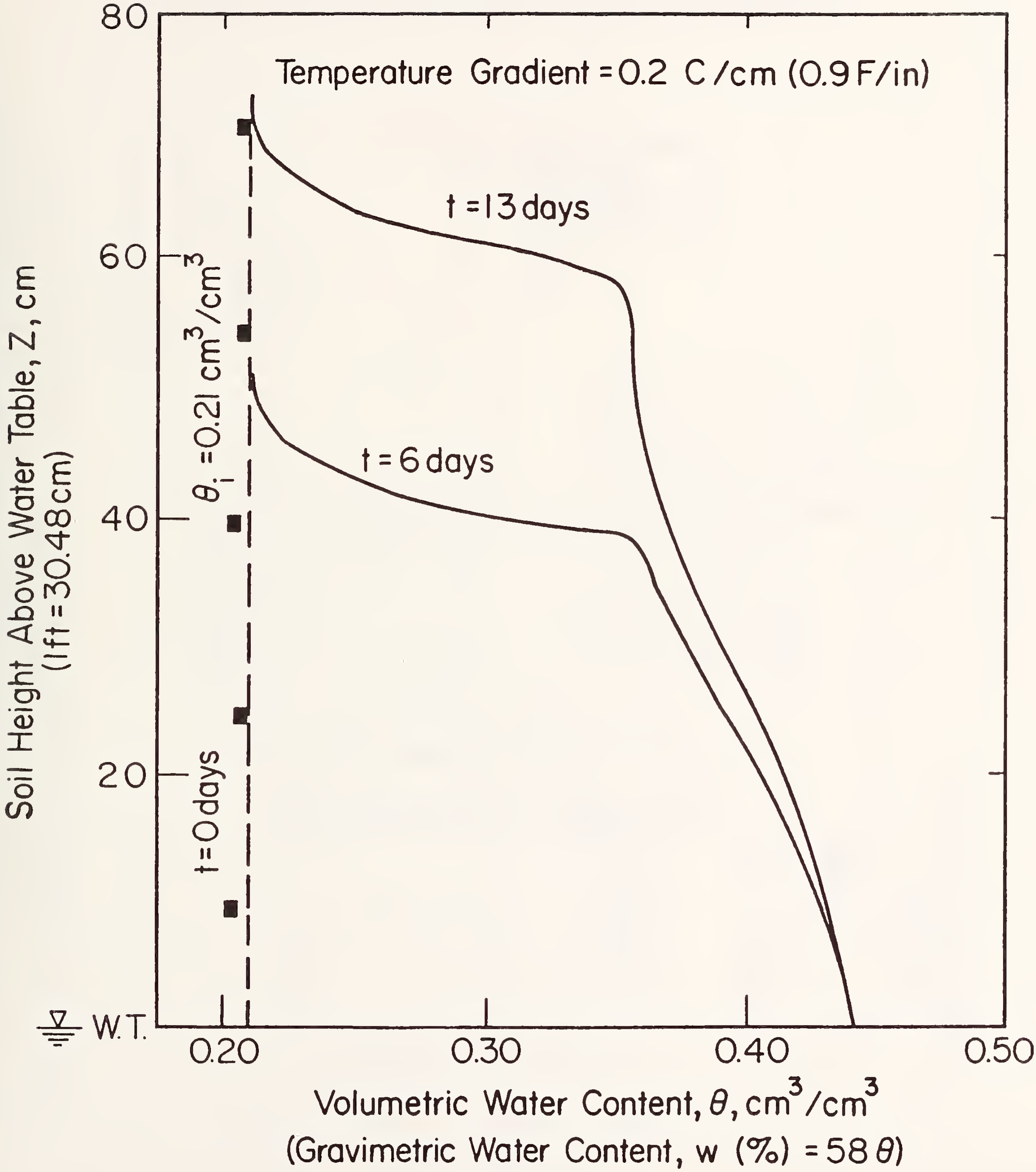


Figure 3.38. Predicted Nonisothermal Moisture Movement for Illinoian Till above Water Table (10).

$$k = \frac{6.214 \times 10^5 (D_{10})^{1.478} (n)^{6.654}}{(P_{200})^{0.597}}$$

$$n = \text{Porosity} = \left(1 - \frac{\gamma_d}{62.4\,G}\right)$$

G = Specific Gravity (gm/c.c.)
(Assumed = 2.70)

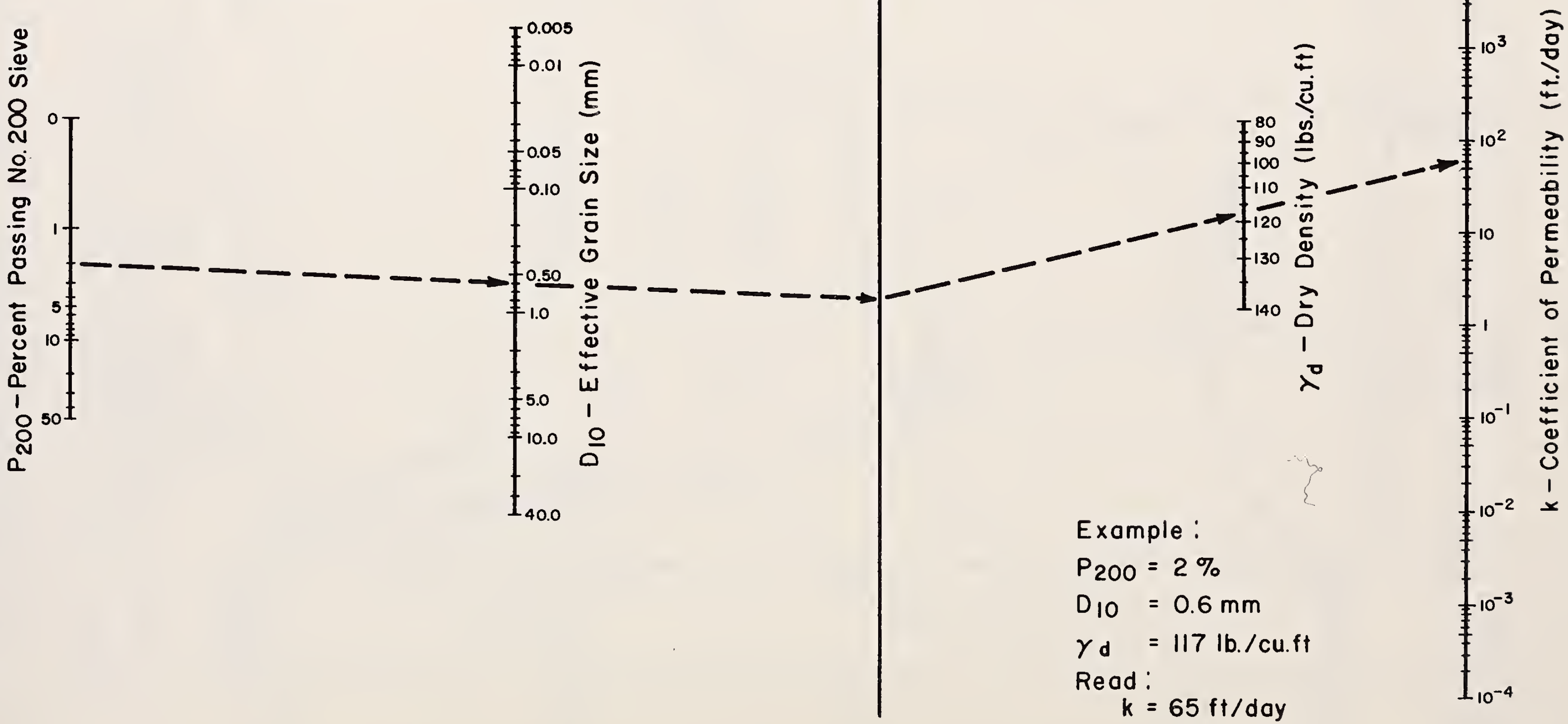


Figure 3.39. Procedure for Estimating Coefficient of Permeability of Granular Drainage and Filter Materials (14).

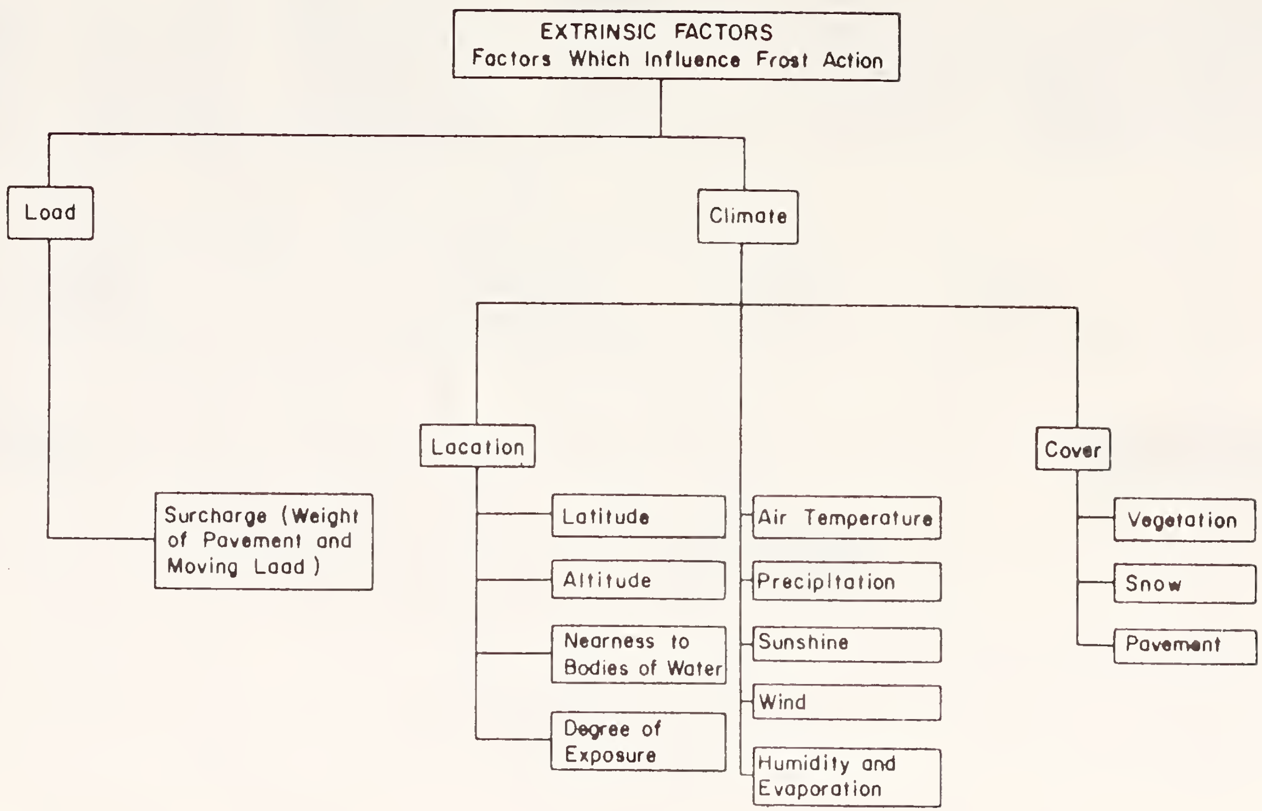


Figure 3.40. Extrinsic Factors Influencing Frost Action (9).

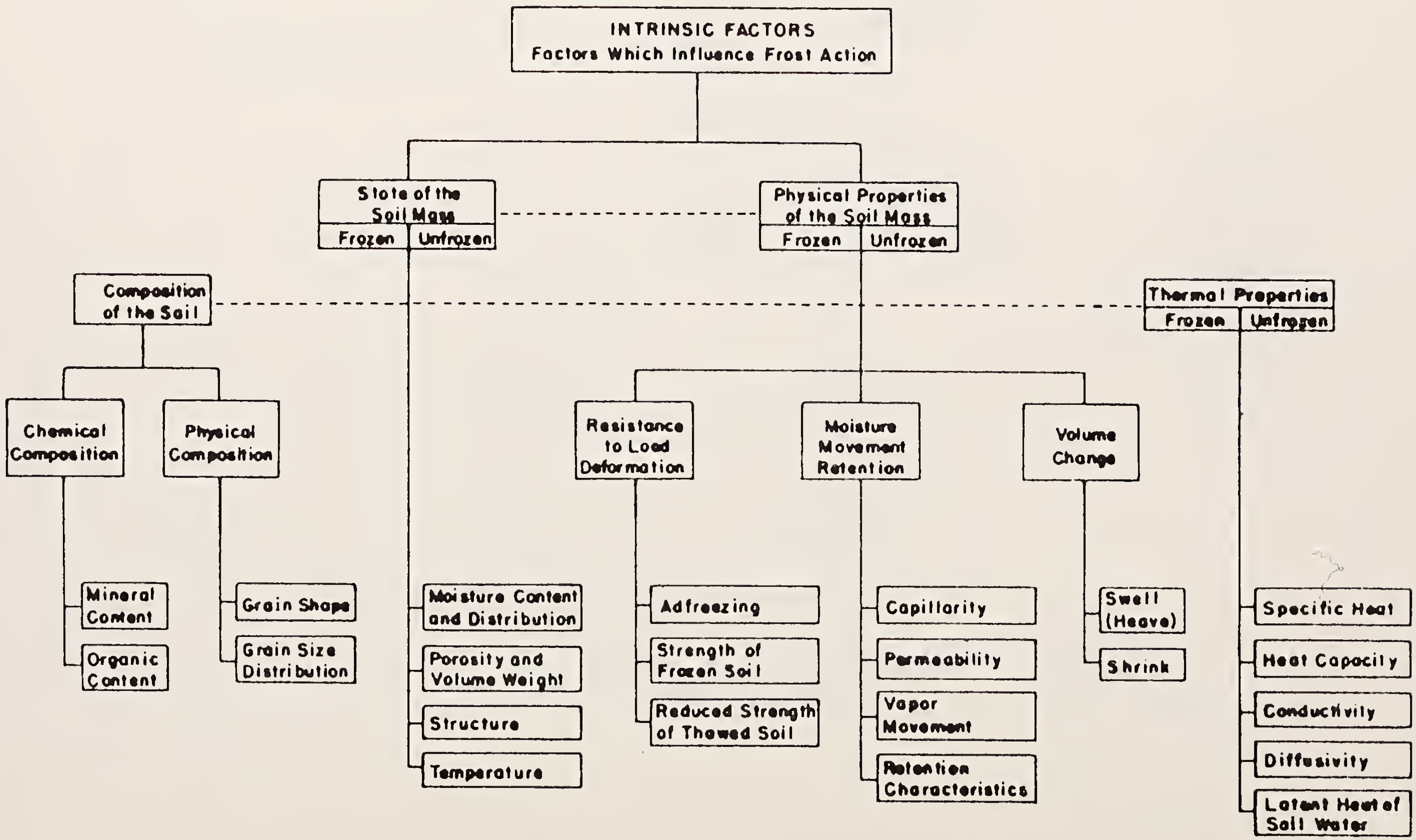


Figure 3.41. Intrinsic Factors Influencing Frost Action (9).

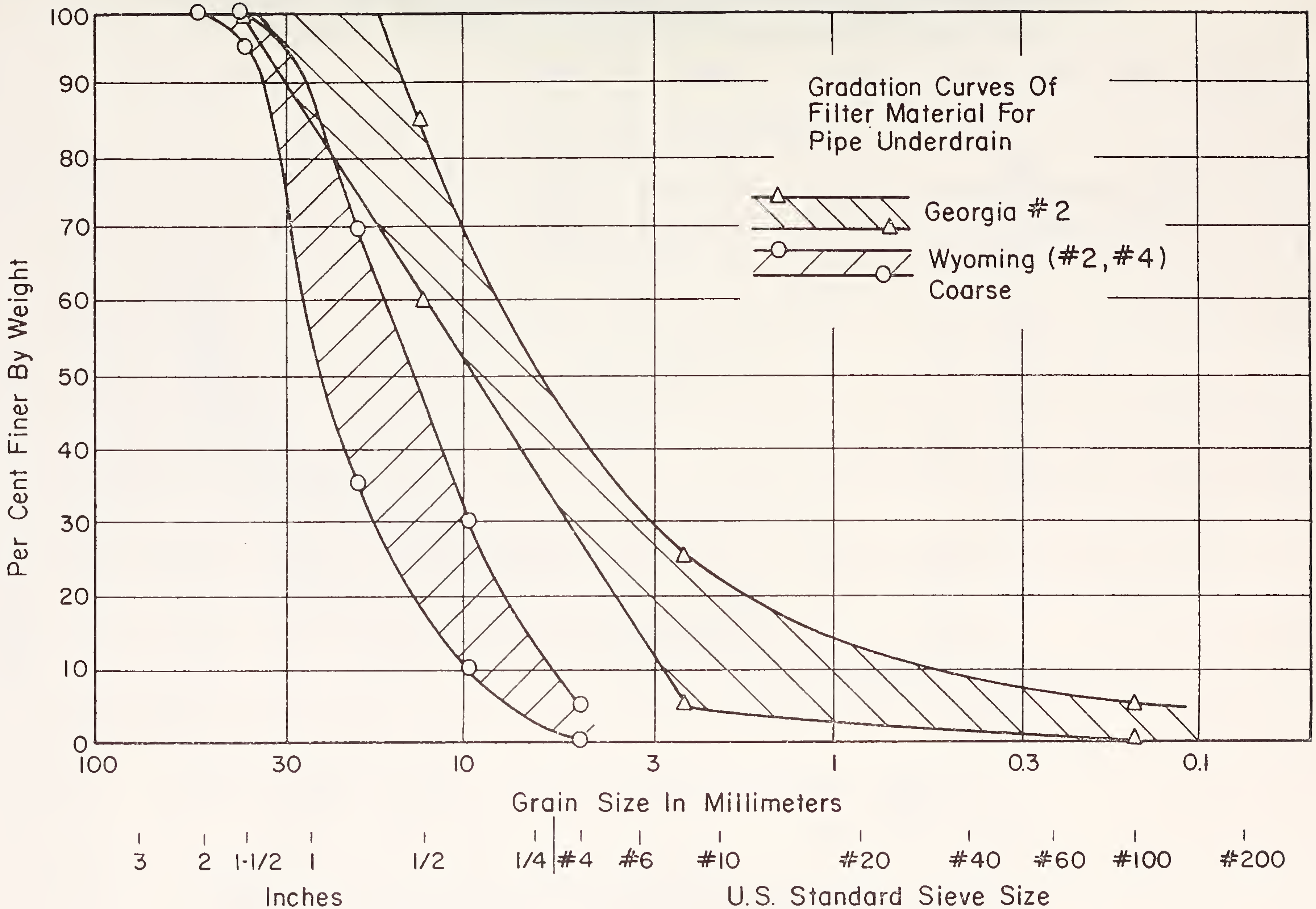


Figure 3.42. Gradation for Filter Material for Pipe Underdrain in Illinois (3).

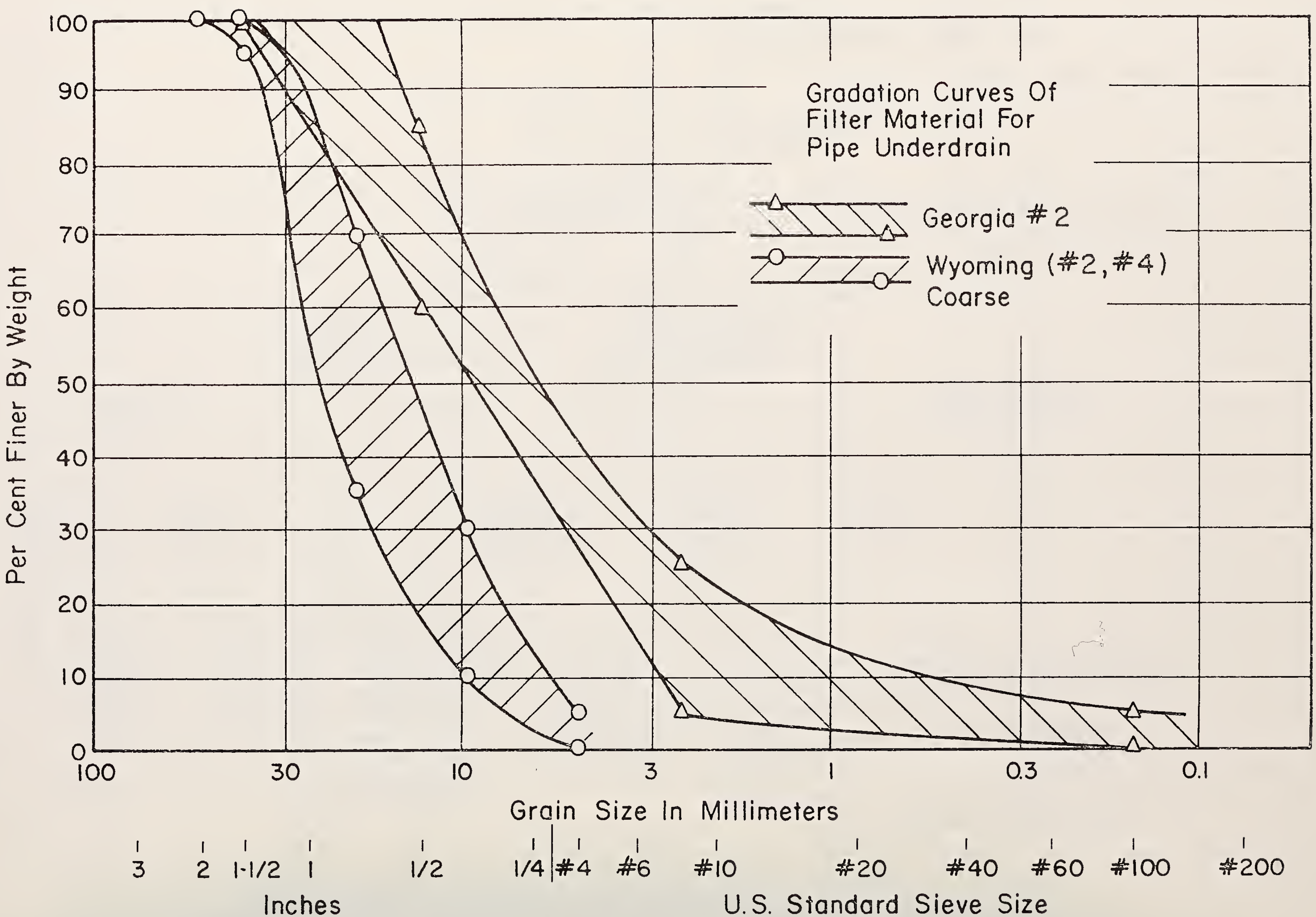


Figure 3.43. Gradation for Filter Material for Pipe Underdrain in Wyoming (3).

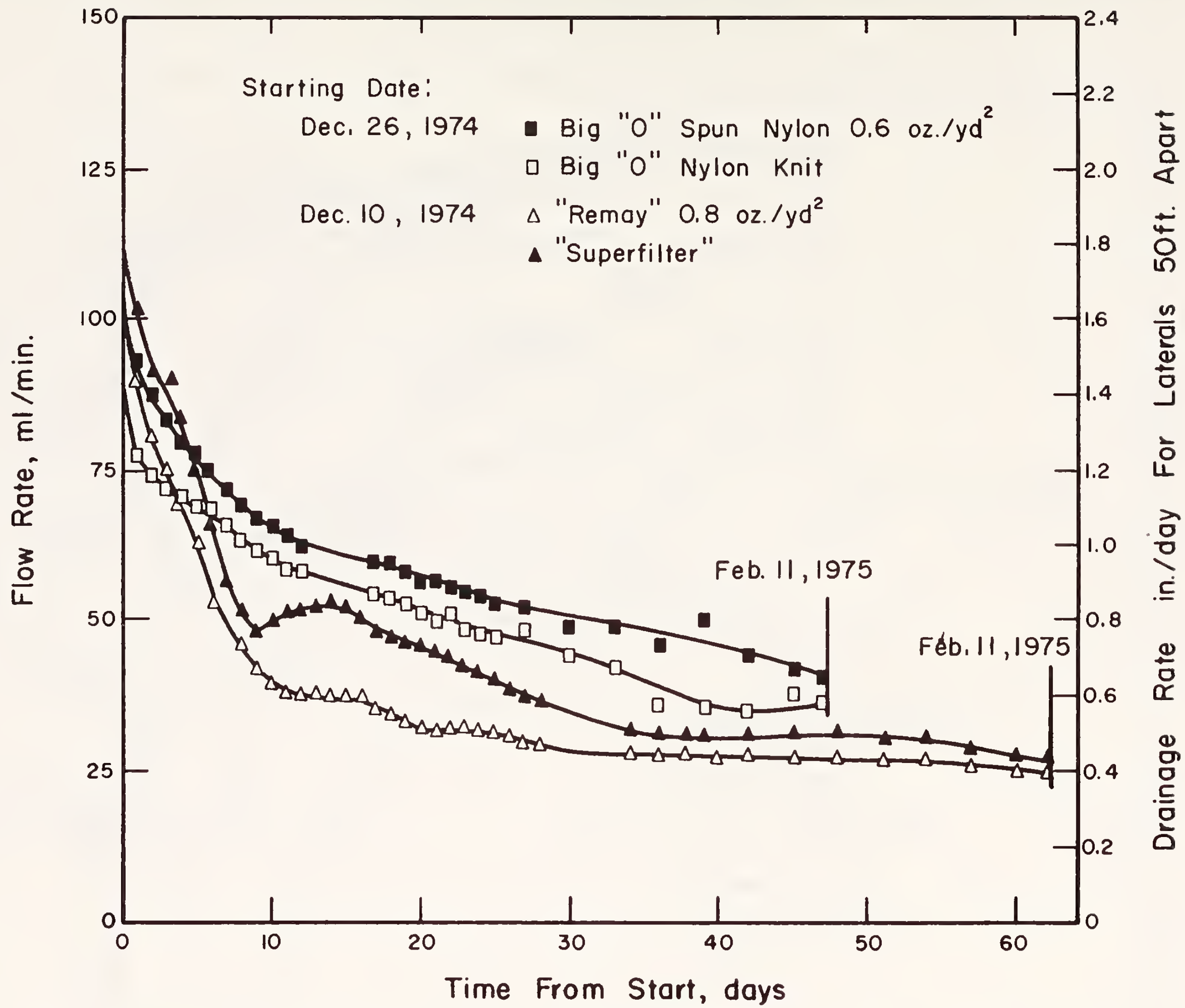


Figure 3.44. Flow of Water Through Drain Tubes Wrapped with Labeled Filter Materials: First Flow Period December 1974 to February 1975 (185).

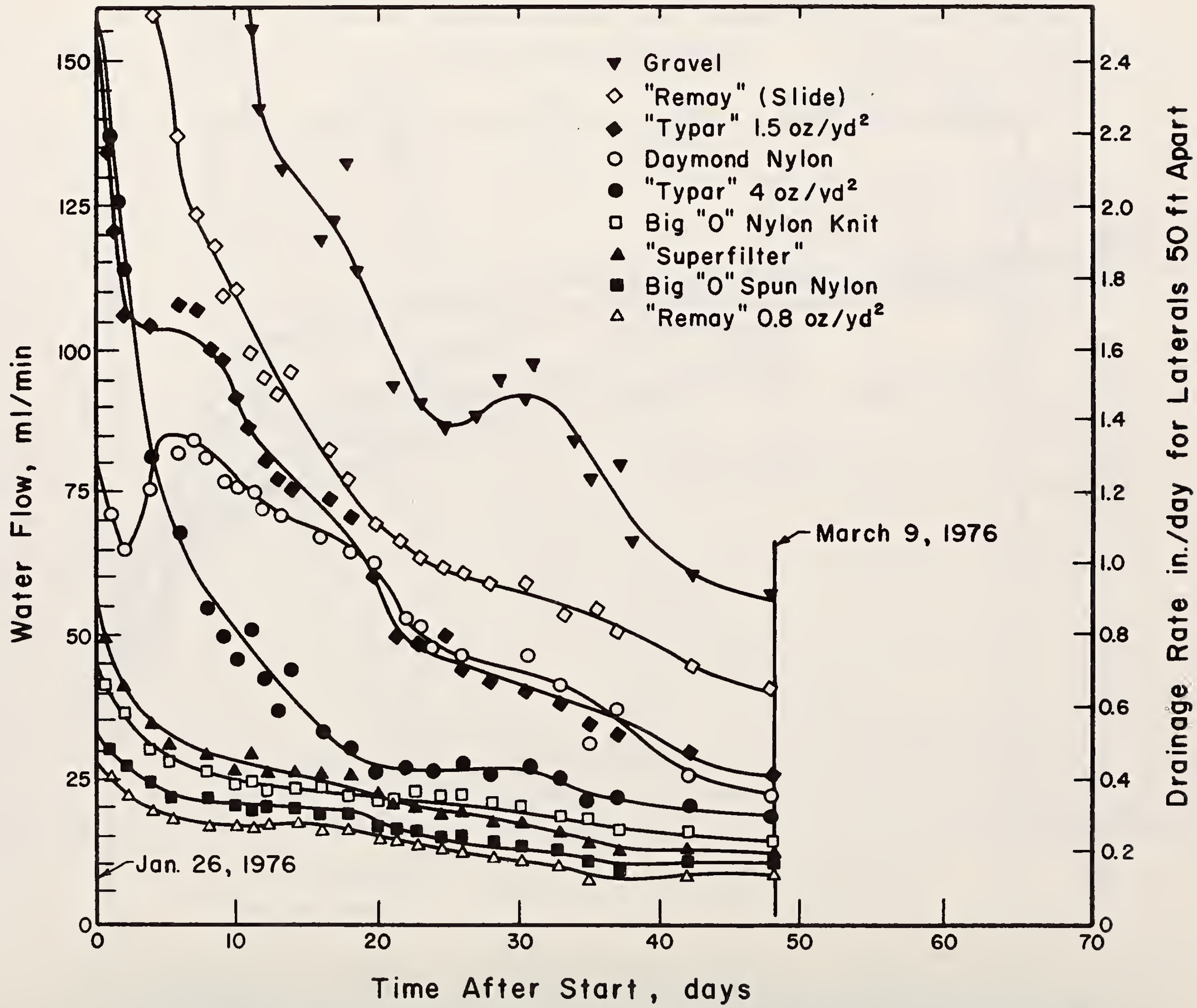


Figure 3.45. Flow of Water Through Drain Tubes Wrapped with Labeled Filter Materials: Third Flow Period Jan. - March 1976 (185).

NOTE: VERTICAL DIMENSIONS OF SECTION
ARE EXAGGERATED FOR CLARITY

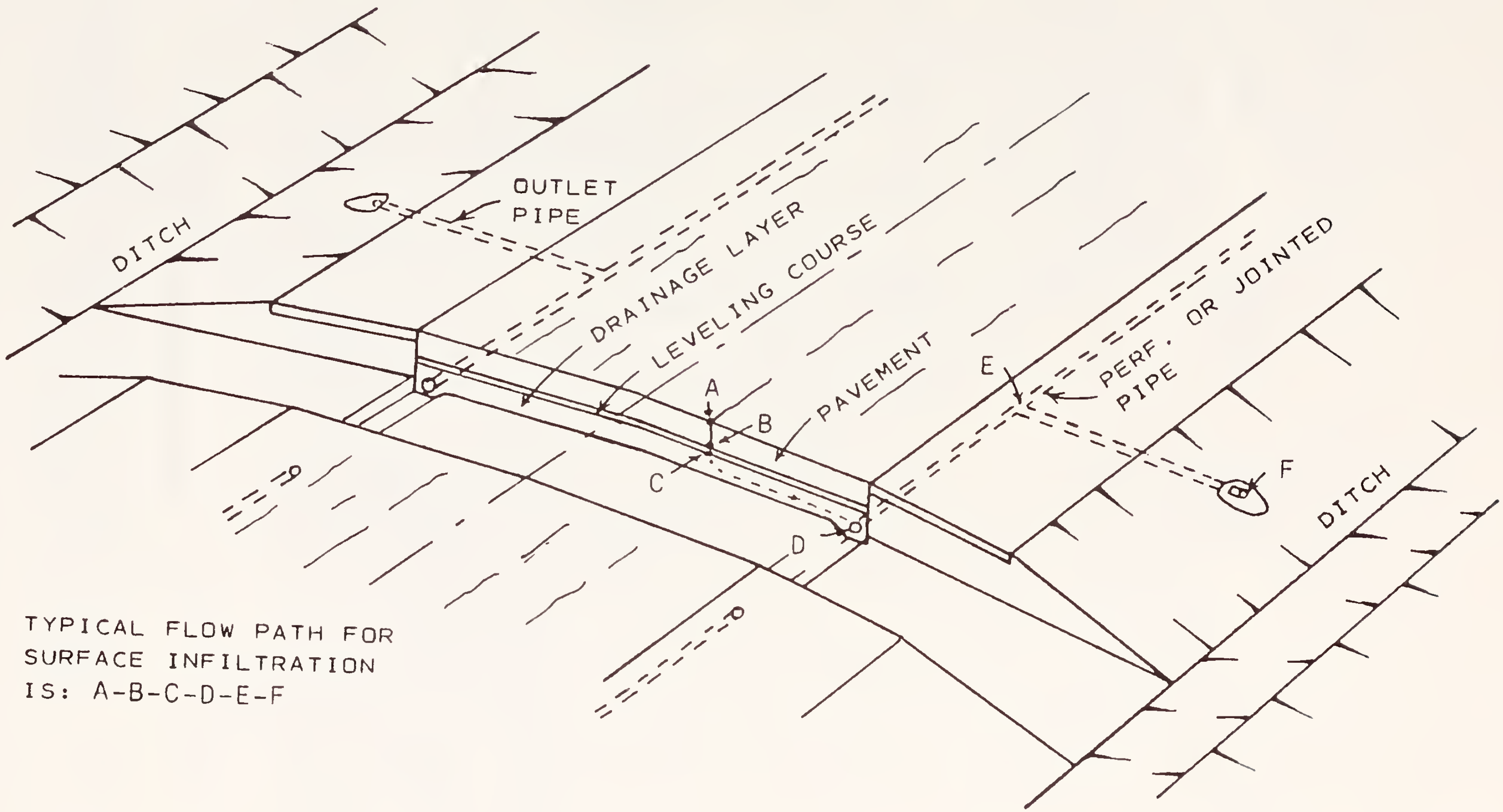


Figure 3.46. Illustration of Flow Path for Condition of Continuity in Pavement Drainage of Surface Infiltration (5).

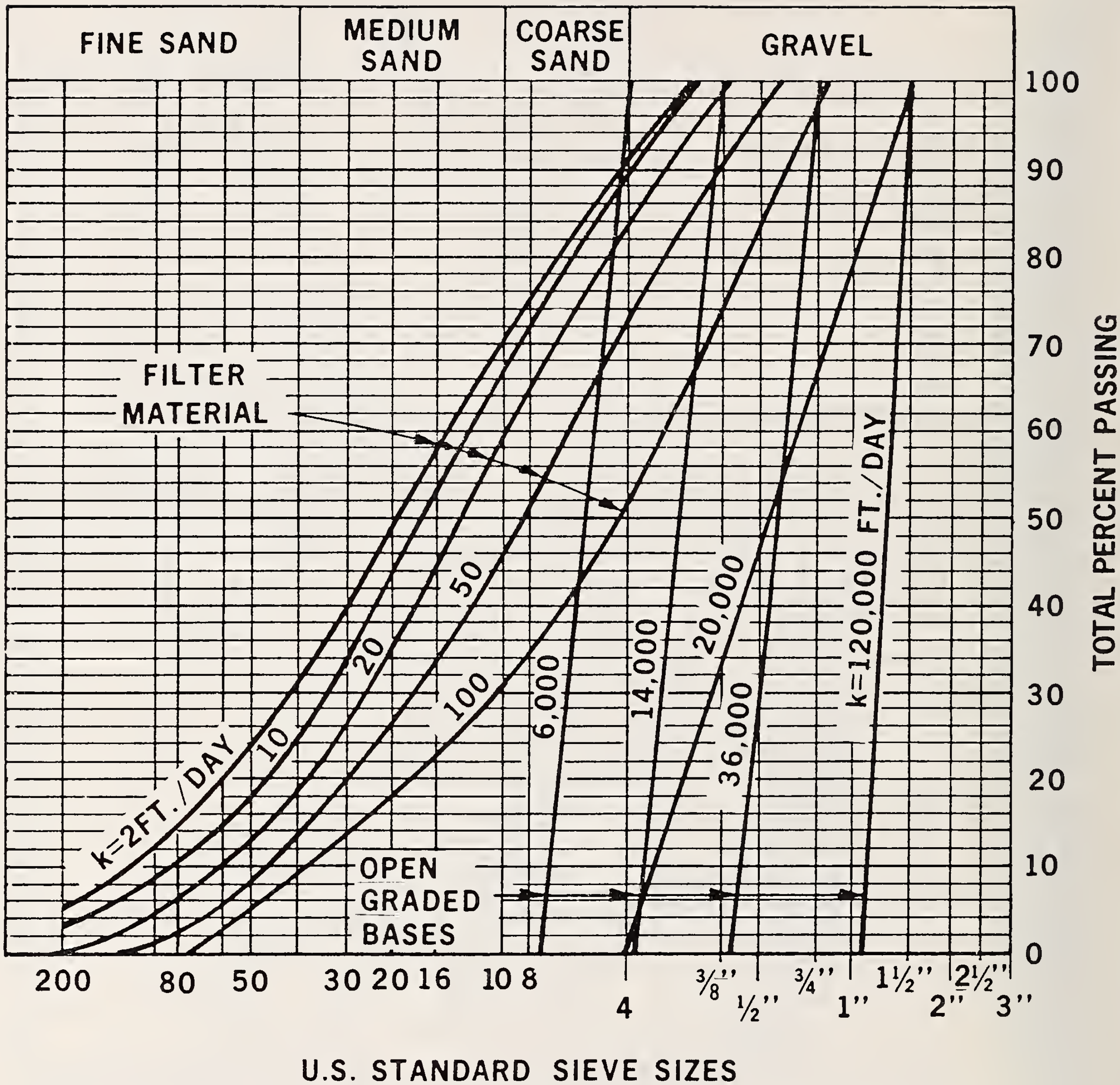


Figure 3.47. Typical Gradations and Permeabilities of Open Graded Bases and Filter Materials (6).

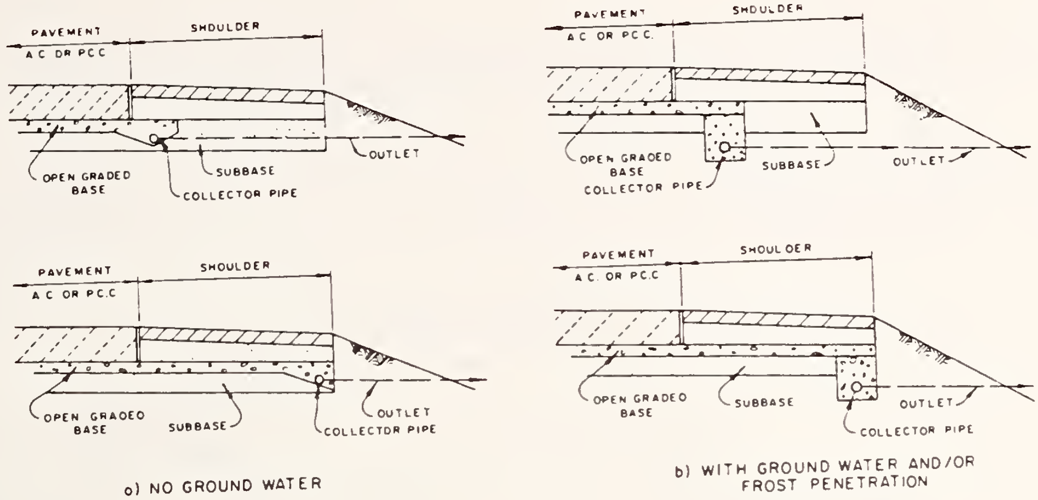


Figure 3.48. Typical Cross Sections of Subdrainage Systems (6).

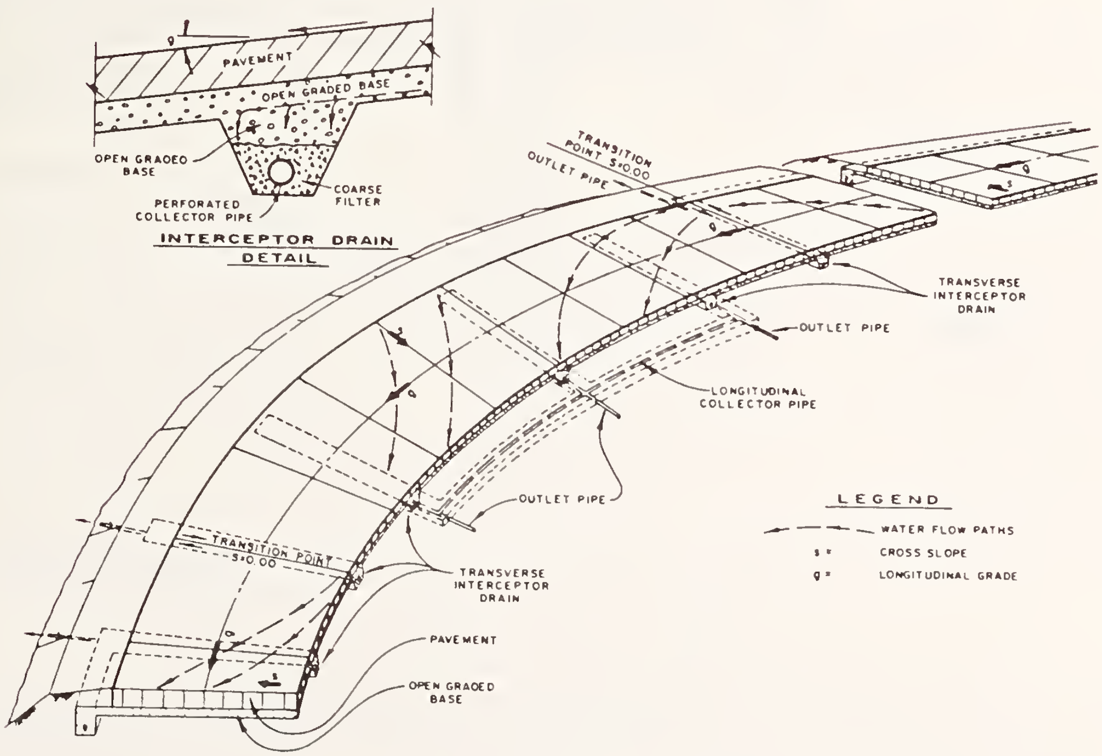


Figure 3.49. Transverse Drains Located on Superelevated Curves (6).

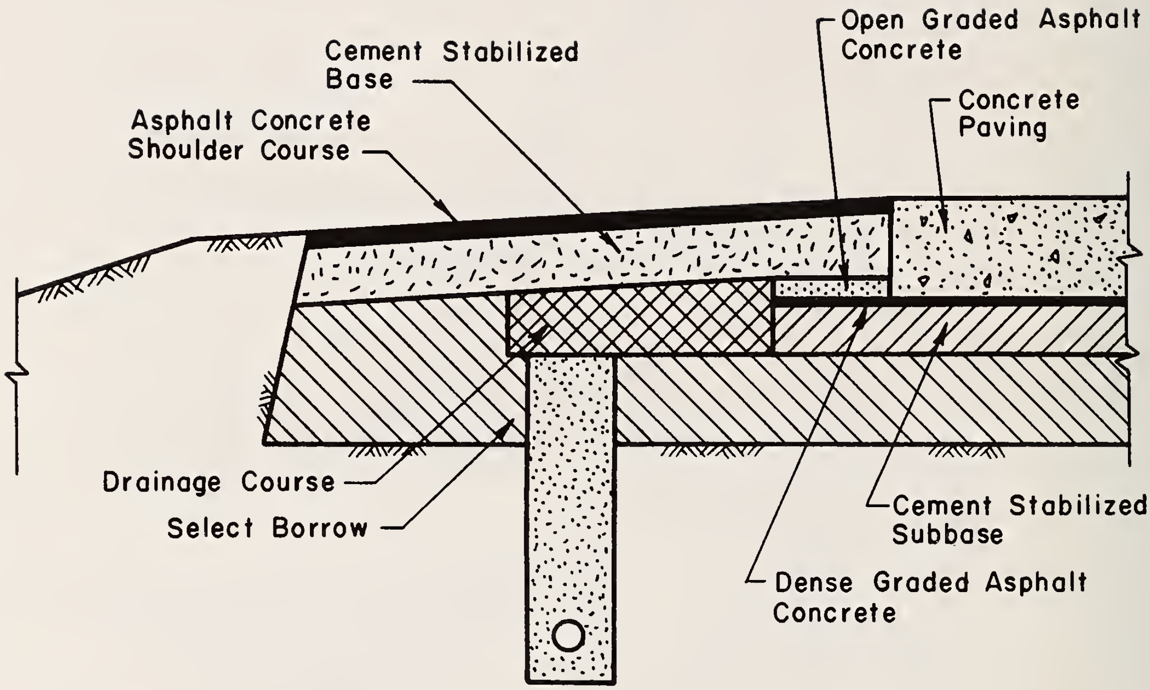


Figure 3.50. Improved Shoulder Drainage System Used in Georgia (13).

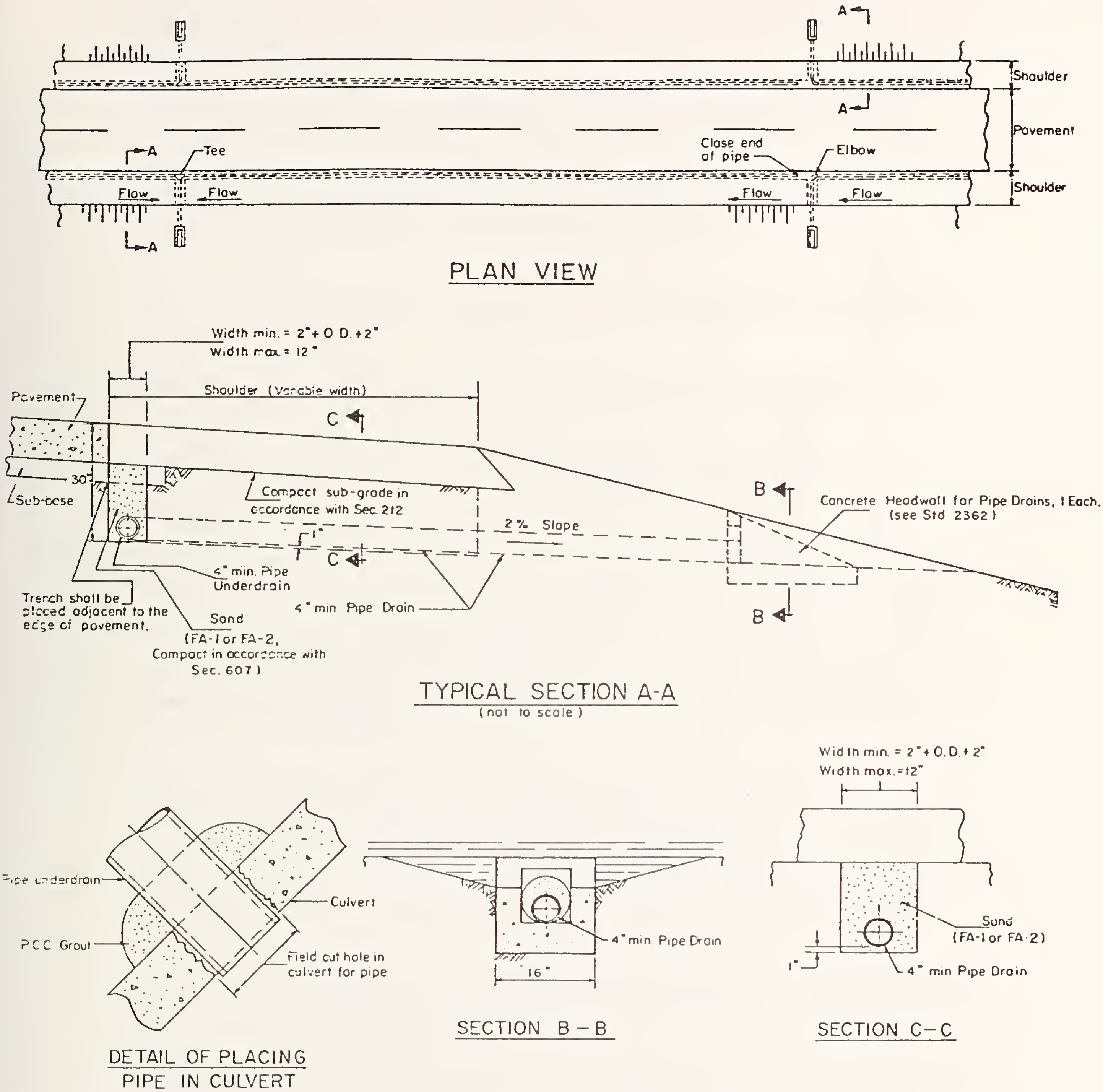


Figure 3.51. Standard Design for Subsurface Drains in Illinois.

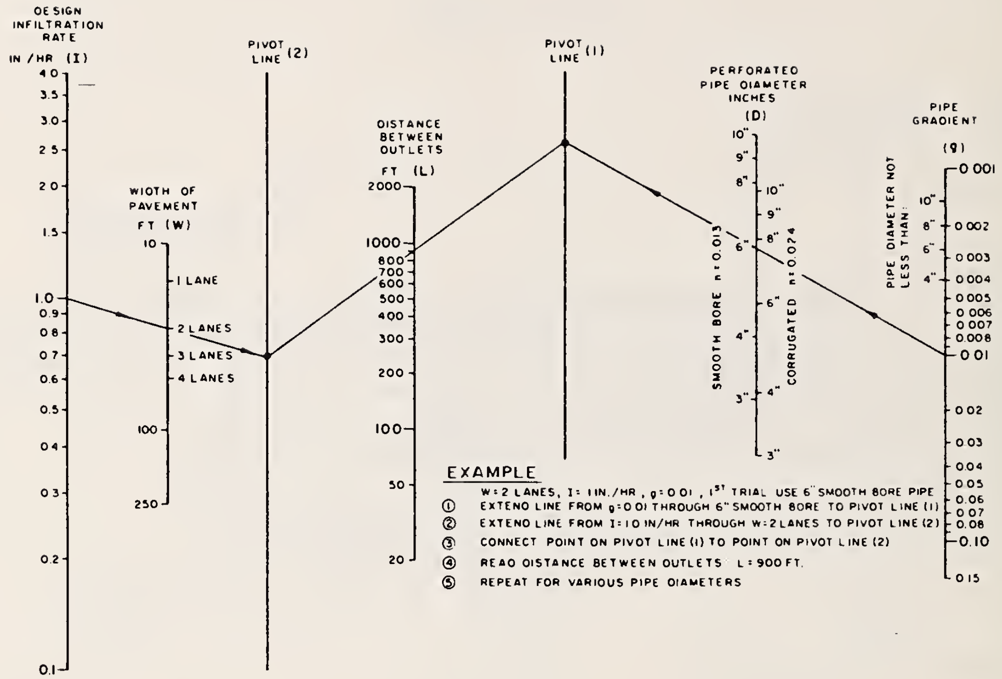


Figure 3.52. Nomograph for Selection of Perforated Pipe Diameters and Outlet Spacings (6).

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Chapter 4

SHOULDERS

4.1 INTRODUCTION

The influence of a shoulder on the performance of a pavement has been recognized for a considerable time; and the term shoulder can be applied to a wide variety of material combinations. The material used in a shoulder depends on traffic level, pavement type and environment; but whatever the structure of the shoulder, there are functions the shoulder must fulfill. These functions are embodied in the AASHTO definition of a shoulder as:

> "--- the portion of the roadway contiguous with the traveled way for the accommodation of stopped vehicles, for emergency use, and for lateral support of base and surface courses. It varies in width from only two feet or so on minor rural roads, where there is not surfacing or the surface is applied over the entire roadbed, to about 12 feet on major roads where the entire shoulder may be stabilized or have an all weather surface treatment" (1).

The two major functions specified in this definition are lateral support and accommodation of vehicles. There are several indirect functions, which are:

1. Removal of water from the mainline pavement,
2. Relieve driver tension over vehicle placement on the roadway,
3. Esthetic value,
4. Provide an extra lane for
    a. vehicle routing during construction,
    b. maintenance equipment,
    c. peak traffic periods, and
    d. slow moving vehicles.

To date there have been only limited studies into the structural adequacy of shoulder design. The most recent study by Barksdale, Hicks, and Emery (2) clearly demonstrates the lack of design procedure, and the inadequacies

of the one procedure in use in California. Structurally, the removal of moisture from the edge of the mainline pavement is critical to the performance of the mainline pavement. The lack of moisture removal at the longitudinal pavement joint is a critical factor in the performance of the shoulder as well.

The necessity to design a shoulder to accept moving traffic and parked vehicles has been considered in the past, primarily from the safety standpoint. These studies have been concerned with how wide to make the shoulder to eliminate the influence of the parked vehicle on the traffic in the traveling lane of the pavement; and the positioning and speed of vehicles in the travel lane as influenced by the width of the shoulder. These factors have largely influenced any structural design of shoulders with experience in use being the prime design factor. In the following sections of this chapter the current design practices will be investigated and loading conditions for shoulders will be discussed. The basic problems in shoulder design that have not been addressed by current design procedures will be discussed and expanded on where necessary. This discussion will point out the necessity of having a design procedure that considers the hydrothermal influence on the performance of the shoulder and materials used in their construction.

4.2 TRAFFIC LOADS ON SHOULDERS FOR DESIGN

4.2.1 ENCROACHMENTS

The shoulders on all highways are commonly built to carry a portion of the mainline traffic, as mentioned earlier. Unfortunately only one state actually designs on this concept. California designs for one percent of

the mainline traffic with a minimum traffic index of 5.0. This one percent of the mainline traffic relates to the expected amount of traffic that encroaches on the shoulder. The loading of the shoulder by traffic encroachments produces a severe strain on the shoulder. Where traffic has been diverted from the mainline pavement onto the shoulder to allow repair of the mainline pavement for even a short time the shoulder deteriorates rapidly, as has been noticed following patching repair of punchouts in continuously reinforced concrete Interstate pavements in Illinois where traffic has been forced onto the shoulder for only four days. The increase in fatigue damage with increased traffic is very distinct even though the traffic level is extremely small compared to a design value of even one percent. The cross sections for these shoulders was eight and three-fourths inches of Bituminous aggregate.

This large influence of a small amount of traffic illustrates the necessity of having a well documented design procedure for the shoulders, which does not exist at present. The survey by Barksdale, Hicks, and Emery (2) on shoulder encroachments in Georgia clearly demonstrated that nearly 2.5 percent of the truck traffic encroached on the shoulder, which would indicate California's design to be very unconservative. The study, however, examined shoulders that provided good contrast with the mainline pavement. The shoulders were bituminous while the mainline pavement was portland cement concrete. Studies have shown that contrast of the shoulders will reduce encroachment (3), so this percentage may still be low. An encroachment study that examines various types of shoulder contrast, will provide a truer indication of traffic encroachment on the shoulder. Such a study should include an investigation of the following:

1. Concrete shoulder on concrete mainline pavement,
2. Asphalt shoulders on concrete mainline pavement, and
3. Asphalt shoulders on asphalt mainline pavement.

This study would show the effect of shoulder contrast in controlling the percentage of traffic encroaching on the shoulders, and would provide, valid design data.

Using the AASHTO Interim Guide equations (4) Barksdale, Hicks and Emery calculated shoulder thicknesses for various material combinations (2), from which examples will be shown later. These calculations clearly show a large change in thickness for increased traffic on asphalt concrete shoulders. There is a similar change in thicknesses for increasing traffic for concrete shoulders. A shoulder design procedure using the AASHTO Interim Guide is a method available at present. The stresses and deflections that develop in a shoulder, however, are not the same as those that develop in the pavement section. The discontinuities caused by material change or cross section changes produce a much more severe condition for the wheel loads which encroach onto the shoulder. To predict deterioration for the complicated cross sections, a stress-dependent finite element study and a field study would be necessary, as will be discussed later.

4.2.2 REDUCING ENCROACHMENTS

Because encroaching loads are such a major problem in shoulder deterioration it would be adviseable to limit or reduce the number of encroachments. This can be accomplished by using contrasting shoulders or delineating the shoulder in some manner like edge striping. The effect of edge striping is illustrated in Figure 4.1 from a study by Jorol (3). This clearly shows that the effect of edge striping is to move the vehicles closer to the centerline. The effect of contrast is shown in Figure 4.2. The average distance from the centerline to the right wheel is nearly two feet less for the contrasting shoulder than for striping alone. The farther away from the shoulder edge

the driver places his vehicle the less likely he is to encroach on the shoulder, and the lower the design percentage of traffic that can be used to design the shoulder.

The studies by Jorol (3) and others (6, 7) clearly show that commercial vehicles always travel closer to the edge of the pavement causing extensive damage to the shoulder. The actual percent of traffic encroaching on the shoulders were not given, although the data might indicate that more than half of the vehicles encroached. These studies do show how to reduce the possibility of encroachments with edge striping and contrast.

Some states that stripe the edge of the pavement place the stripe some distance off the shoulder edge. This practice is to save on painting expenses because, presumably, the paint would not be worn off by traffic as it would when placed on the pavement edge. In reality, this may be a false economy. A majority of drivers will use the edge stripe as their indication of the pavement edge (6). This will be particularly true at night when the actual pavement joint edge is indistinguishable, even with a high contrast shoulder. Placing the stripe on the shoulder may actually cause more encroachments and produce a more rapid deterioration than placing the stripe on the pavement.

Recalling that even four days of traffic on the shoulder can severely deteriorate the shoulder it appears obvious that everything possible should be done to minimize the traffic on the shoulder. Figure 4.3 illustrates the effect edge striping has on vehicle placement (6). The presence of the edge stripe, and its position on the roadway relative to the shoulder does influence the placement of the vehicle. Drivers follow the edge stripe.

For shoulder-pavement combinations where there is good contrast, such as asphalt shoulders on concrete pavements the use of an edge stripe may be

questioned since the contrast serves to keep the traffic away from the shoulder more effectively than edge stripes. The easing of driver tension at night, however, makes stripes useful. For concrete shoulders on concrete mainline pavement the effect of encroachments has not been shown as conclusively as for asphalt shoulders. If the shoulder is designed with a smaller structural cross section, however, it is adviseable to keep the traffic away from the reduced section as much as possible.

The problems associated with very thin structural sections are quite obvious. Typically the outer edge of the shoulder is very thin because the structure is designed thickest where the majority of the encroachment will occur. This typically results in an underdesigned section that may fail under the first truck that pulls off onto the shoulder. This damage can be related to moisture, since in the initial period of the design life the properties will be near their design values and will support a load. If the granular material in the shoulder does not drain properly, as discussed in a previous chapter, the strength will decrease and the load the shoulder previously supported will cause the shoulder to fail. This problem is very critically related to the effects of temperature and moisture on the pavement system, as will be shown in the following sections.

4.3 GENERAL SHOULDER TYPES

4.3.1 CURRENT DESIGN PRACTICE

The shoulder designs in current use have developed primarily by trial and error (5). The two major shoulder types are portland cement concrete and asphalt concrete shoulders. A portland cement concrete shoulder is a rigid pavement slab placed on a prepared material after the mainline pavement has been placed. These shoulders, no longer considered experimental by the FHWA,

when placed adjacent to rigid pavements, may or may not be reinforced and may or may not be anchored to the main line pavement (8). There are three distinct types of asphalt concrete shoulders. The bituminous surface-treated shoulder is composed of a gravel shoulder treated with liquid bitumen to produce a surface mat at least an inch thick. The bituminous aggregate shoulder consists of a high quality bituminous mat placed over an aggregate base course. The full depth asphalt shoulder uses asphalt mixtures in all layers placed on the subgrade (8).

When the design cross section for the mainline pavement extends completely across the paved surface with no change whatsoever, the shoulder portion of this structure is not subjected to the same damage potentials as sections which have structural changes. These full width sections are not considered as shoulders in the discussion in this chapter. These sections should not be confused with regular shoulder sections that have been overlayed from edge to edge, giving the appearance of full width paving. The old shoulders will still be structurally different from the mainline pavement.

Each shoulder type listed above may be further differentiated by the type of base course used in its construction. The following base course material combinations have been tried with varying degrees of success:

1. Granular base,
2. Bituminous stabilized base,
3. Cement stabilized base,
4. Lime stabilized base, and
5. Pozzolanic aggregate mixture.

Some of these materials have met with more success than others as will be discussed later.

There have been two very recent surveys conducted by Portigo (8), and Barksdale, Hicks, and Emery (2), that illustrate the current status of shoulder design. Protigo determined the following:

1. Only fifteen states have documented policies regarding shoulder design.
2. Twenty-eight have no documented policies, but have shoulder paving standards. Six of these states evaluate individual projects before making decisions on paving.
3. Five states pave the shoulder integrally with the mainline pavement.

California is the only state with a design procedure for their shoulders (2). The shoulders are designed for one percent of the mainline traffic with a minimum traffic index of 5.0. Iowa designs its shoulders for the maximum wheel load, which presumably is the same design as the mainline pavement (9).

As it appears most states do not have a set policy for design of shoulders, the process of trial and error and engineering experience will have determined the shoulder designs that have developed in most states. Hicks, Barksdale, and Emery (2) conducted a survey of asphalt concrete shoulder designs which is reproduced in Table 4.1 through Table 4.6. From these tables it can be easily seen that there is quite a large variation in the shoulder sections throughout the country.

The thicker shoulder sections appear in the areas of the country where cold temperatures or excessive moisture are found. This is illustrated in Figure 4.4 which shows the total thickness of processed or stabilized material

plotted for each state with the surface thickness indicated in parenthesis. The distribution shows that engineers, over a period of time have recognized the effects of the environment even though they have no rational design procedure.

The design sections for portland cement concrete shoulders are given in Table 4.7 (2). The environmental effect on a concrete shoulder is not as severe as it is for an asphalt shoulder, however, it is interesting to note that tie bars are confined primarily to the Northern States where frost heave is a severe problem. Texas, has a serious problem with expansive clays which produce severe distortion in a pavement structure, similar to frost heave; while Georgia has severe problems with faulting (10).

The use of the AASHTO interim procedure (4) to design shoulders is questionable and should not be adopted without further study although it does present an approach, the boundary conditions at the pavement and shoulder edge produce a stress state that it entirely different from that caused by a wheel in the wheel path of the mainline pavement. With these different stresses and deformations, the rate of deterioration cannot be expected to be similar to what was measured on the AASHTO road sections, even for shoulders in the same climatic region as the AASHTO sections. Further, the shoulder could have moisture concentrations at each edge, as will be discussed later, which make the behavior of the shoulder even more complicated. Asphalt concrete shoulders are influenced much more by these factors than are the concrete shoulders as the asphalt concrete is more sensitive to fatigue damage from material property variation.

4.3.2 MATERIALS

The use of various materials to eliminate problems in the shoulder have not always been successful, and in some cases may have caused more serious

problems. A study of cement and pozzolanic aggregate mixtures used in Illinois shoulders verified this (11). These two materials deteriorated badly when exposed to freeze-thaw cycles in the presence of a salt water solution. De-icing salts that collect along with water in the longitudinal pavement shoulder joint produce such a solution in direct contact with the susceptible materials. Freeze-thaw cycling resulted in expansion of the shoulder and a breakdown in the cementing products.

When proper materials are utilized that are resistant to normal environmental deterioration, the construction and design of the shoulder may still produce a situation that is damaging to the pavement as well as the shoulder. A common design for a continuous reinforced pavement in Illinois is shown in Figure 4.5. The slab is underlain by a bituminous aggregate mixture that extends 18 inches into the shoulder. The shoulder is also a bituminous aggregate mixture, commonly up to 11 inches thick. This design holds water in the pavement system much longer than shoulder designs using granular layers or drainage blankets. This ponding of moisture keeps water in contact with the concrete longer allowing more to be absorbed into the concrete. This accelerates such problems as "D" cracking and steel corrosion. This accumulation of water will also hasten pumping and/or hydraulic abrasion of the bituminous aggregate mixture both in the shoulder and under the pavement slab. One of two things is necessary to eliminate the moisture problem under the pavement. Either the water must be given a free path out of the pavement system or the water must be prevented from entering the pavement system to begin with. These problems apply to all pavements since, as was shown earlier, most paving materials are not free draining.

4.3.3 LONGITUDINAL JOINT PROBLEMS

It should be evident by now that the major problem in shoulder design, apart from traffic loadings, is the integrity of the longitudinal shoulder joint. This is the point where the majority of water will enter the pavement system and begin its damage process. Concrete shoulders are tied to the mainline concrete pavement in a number of states (2). This maintains good joint integrity when the shoulder is placed subsequent to the mainline pavement. An asphalt concrete pavement placed next to the portland cement provides no way to obtain a tight joint. The thermal property difference in the two materials automatically provides differential movement at the joint (5). The material difference provides no means of physically tieing the shoulder to the mainline pavement. Sealants, discussed in the chapter on maintenance, are at present ineffective in providing complete and permanent moisture proofing of the joint.

Full width paving, using one material, concrete or asphalt, with no shoulder joint provides the easiest method for eliminating moisture infiltration at the joint. Striping would then deliniate the portion of the pavement to be considered as the shoulder. Since this is not always a practical alternative, drainage should be provided for in the shoulder to remove the water entering the longitudinal joint before it can soak into the foundation material where it cannot be removed by conventional drainage procedures.

4.3.4 LAYER AND MATERIAL ARRANGEMENTS

The strains or stresses in the surface layer will affect the fatigue performance of that layer. The inclusion of pipes, drainage blankets, filter materials all will tend to alter the stress distribution in the shoulder even farther away from that of a normal pavement section. Drainage may be added to shoulders even after construction, which makes it very popular at

present to add edge drains. Figure 4.6 shows some typical improved shoulder sections incorporating mini-drains (12). The presence of pipes near the surface and the large number of layers illustrates the complexity of the situation and the need for thorough computer and material characterization to fully describe the behavior of the shoulder.

These designs may cause problems also. When the mini-drain is located some distance from the shoulder pavement edge the filter layer beneath the drain will collect and store water since the drain is at a higher elevation. Unless considerable slope is put in the shoulder layers, this water may pond and act as a moisture source for the subbase and subgrade. Figure 4.10-b shows an edge drain placed some distance away from the shoulder pavement edge. This arrangement exposes the infiltrating water to a much longer path than a drain placed right at the shoulder's edge. This longer path brings the water into contact with more material which increases the likelihood that the material will take on at least some of the water. This small amount of water may be enough to alter the performance and hasten the fatigue of the shoulder.

The more common procedure is to place the edge drain right at the longitudinal shoulder joint to minimize the amount of time the water would be in contact with foundation material. A typical installation in shown in Figure 4.7 for an Alabama highway (13). Because the drain involves reconstruction of the shoulder at its most critical point, the construction of these drains is critical to the performance of the shoulder. The material placed in the trench must have the proper drainability characteristics to remove water and not act as a source. The material placed in the trench must also have the necessary strength and deformation characteristics to perform satisfactorily under traffic.

The compaction of the drainage material is very important in regards to structural behavior of the shoulder. Improperly compacted backfill material in the trench can cause several problems. Consolidation of this material, particularly sands, causes surface settlement that will crack the asphalt shoulder providing more area for water infiltration. Consolidation of the backfill material will also decrease the permeability, defeating the original purpose of the edge drain. Consolidation in the laterals will provide spots for ponding and will accelerate erosion along the outlet.

More conventional appearing methods to remove water from the longitudinal joint have utilized drainage blankets, typically of open graded asphalt concrete, which has met with varying degrees of success (2, 14). Figure 4.8 and Figure 4.9 illustrate some common shoulder sections in use in the United States, some of which utilize drainage blankets. Even in these structures the analysis of potential performance is complicated by the arrangement and type of materials used. At present there is not enough test data available to perform an analysis of behavior such as fatigue cracking and rutting as is done in the VESYS II-M program (15). An indepth finite element study using stress dependent properties would give an indication of the shoulder performance relative to data commonly seen for mainline pavement sections. Once the geometrical variations have been investigated to determine their influence on performance, design recommendations can be made.

These design recommendations must take into account seasonal and long term moisture changes. The moisture state will affect both the strength and deformation characteristics of the foundation material. Because most shoulders will allow water free access to the foundation material the change will be greatest at the shoulder pavement joint where performance is most

critical. The effects of moisture on fatigue life, as utilized in VESYS and JCP-1 (15, 16) computer programs for pavement design will be more severe at the shoulder, and the moisture states must be predicted to allow fatigue damage to be obtained. The following section will illustrate the effects of moisture and temperature on shoulder performance.

4.4 SHOULDER PROBLEMS: HYDROTHERMAL INFLUENCES

4.4.1 GENERAL

Shoulders have presented problems since they were conceived. Even though more appropriate designs have been developed over time, damage still prevails in the shoulder as has been demonstrated in the previous sections of this chapter. Gravel shoulders provided no removal of moisture from the pavement edge and lost support when moist, necessitating continual maintenance. Paved shoulders were supposed to reduce the maintenance effort and to improve the performance of the pavement by adding lateral support and removing moisture from the pavement edge. However, it is evident that moisture is not being removed in existing pavements as exhibited by the large amount of distress present.

Different materials exhibit different moisture retention properties as discussed in an earlier chapter; and even with adequate drainage some change in moisture is likely to take place. This change will be at the pavement shoulder joint, which seldom possesses enough integrity to restrict the entry of water (5). This moisture will reduce the strength and alter the deformation characteristics. Changes in these properties will seriously affect the performance of the shoulder more so than the pavement, due to the reduced section in the shoulder. The reduced thicknesses will also affect the temperature distribution which affects the deformation characteristics of the materials. This section attempts to show the affects moisture and temperature will have on the shoulder by illustrating the affects these two

factors have on materials used in shoulders. Most commonly the granular and bituminous materials exhibit the largest change in behavior, and shoulder settlement and fatigue are the two most common forms of distress that result. Typical examples of these distress types can be seen along all major highways in varying degrees of severity.

4.4.2 GRANULAR MATERIALS

The resilient deformation of the granular materials will influence the fatigue behavior of the surface material. The residual, or permanent, deformation-rutting will affect the overall settlement of the shoulder structure. Moisture has a large influence on the resilient behavior of granular material, the resilient modulus (applied stress divided by the recoverable strain). Moisture content, saturation, or volumetric moisture content have been related to the resilient modulus (7). Soil moisture suction has recently been used with much success to relate moisture and environment to the behavior of pavement foundation material. Soil suction relates moisture and clay mineralogy as shown in Figure 4.10 and Figure 4.11 (19, 20). The higher the suction, the lower the moisture content for a given material, and the higher the clay content the higher the suction for a given moisture content. Figure 4.12 shows the change in resilient modulus for different suctions (18). The lower suctions, (high moisture content), produce a low resilient modulus, or a high resilient strain. This will produce accelerated fatigue in the asphalt surface.

Temperature also produces a change in the resilient modulus, as shown in Figure 4.13 (18). The resilient modulus at 22°C is taken as the reference value. An increase in temperature produces a decrease in the resilient modulus (a larger resilient strain), while a decrease in temperature increases the resilient modulus (a smaller resilient strain). Thus, fatigue damage can

be more severe during the summer months, even when the effects of moisture are ignored. The effect of soil type is also shown with the pastic clay giving the poorer performance. During the winter months the fatigue will be less because the resilient modulus will be greater, the resilient strain less.

The residual strain is the permanent strain in the material, and it controls the rutting, or settlement of the shoulder edge. The effect of moisture, or soil suction, on the residual strain is shown in Figure 4.14 (18). The higher the moisture content the lower the suction the larger the residual or permanent strain. This permanent strain also gets larger with more loadings on the material, as shown in Figure 4.15 (18). The effect of temperature on the permanent strain is given in Figure 4.16 (18). An increase in temperature from 22°C to 39°C will increase the residual strain from 1 to 4 times depending on the soil type. Thus the potential for settlement is greater in the summer when the temperatures are higher. Any addition of moisture through the longitudinal joint will only accelerate the process as discussed.

4.4.3 CONCRETE

These environmental problems also have the potential to effect concrete shoulders in the same manner. The stresses transmitted through the slab will be smaller than for flexible shoulders, but the formation of voids due to permanent deformation will still be greater in the warm weather and will be accelerated by additional moisture entering the longitudinal joint. The formation of this void will accelerate pumping under the shoulder and will provide a beginning for erosion to progress under the pavement slab.

4.4.4 BITUMINOUS MATERIALS

In bituminous materials the effects of temperature are more pronounced than moisture effects due to the waterproofing of the asphalt; and the effects of moisture are somewhat recoverable upon drying (21). The temperature sensitivity is illustrated in the viscoelastic nature of bituminous mixes, requiring tests at varying temperature levels. Figure 4.17 shows rut depth as a function of the rate at which rutting accumulates (22). These data are from test tracks under controlled conditions, and the effect of temperature on rutting for this one pavement is obvious. The higher the temperature the faster rutting will accumulate. When the rate of rutting is the same, a higher temperature will produce more rutting, or settlement along the pavement shoulder edge.

The resilient modulus will affect the deformation in the shoulder and directly affect the fatigue performance. Temperature exerts a large change as shown in Figure 4.18. As the temperature increases, the resilient modulus decreases which means the resilient strain is increasing. The larger the strain, the more rapidly fatigue damage will occur. Figure 4.19 illustrates the effect of freeze-thaw cycles and moisture on the resilient modulus of an asphalt-emulsion-treated mix (21). Freeze-thaw cycles on a dry mix did no damage and even strengthened the mix slightly. Upon vacuum saturating, however, the resilient modulus decreased significantly and subsequent freeze-thaw cycles demonstrated a clear trend in further decreasing the resilient modulus, producing more rapid occurrence of fatigue damage. Drying, following 50 freeze-thaw cycles, however, brought the resilient modulus back to the pre-soaking level. This emphasizes the importance of insuring proper drainage for bituminous treated bases under the mainline and shoulder where moisture could accumulate. Unlike a granular material, if properly drained the damage will not be permanent in a bituminous mixture.

4.4.5 SEVERE ENVIRONMENT-FROST HEAVE

This discussion illustrates how the climate can severely effect the performance of a shoulder and should be considered in a more detailed manner in future design procedures. While the influence of temperature and moisture can be indicated and the moisture condition predicted by environmental parameters, as discussed previously there is still the effect of a severe environment to consider. Frost heave in the Northern areas presents a severe distress for the surface and base course of the shoulder. This distress will be accelerated when the pavement-shoulder joint allows moisture to accumulate. Due to material differences and smaller thicknesses, the temperatures in the shoulder may be lower than those under the pavement, and frost may penetrate deeper, faster. As the frost line penetrates the pavement and shoulder, the heaving of frost susceptible material will produce a larger deformation in the shoulder where the overburden pressure of the shoulder is lower than that of the pavement. When there is no structural integrity between the pavement and the shoulder, the shoulder will rise above the pavement edge trapping runoff and directing it into the pavement structure. When moisture is accumulated there will be even larger volume changes and resulting distress. The inclusion of drains will further alter the freezing characteristics under the shoulder, possibly freezing the drain pipe area while the surrounding material remains unfrozen.

Given the fact that moisture will concentrate under the outer edge of the shoulder as well as at the longitudinal joint, the problem of the frost heave is even more serious for the shoulder area. The concentration of moisture in these areas will produce more heave than in the other areas. This disparity in the deformation across the width of the shoulder produces severe stress concentrations in the shoulder area, particularly for asphaltic

concrete shoulders. This action may produce the alligator or fatigue cracking at both edges of the shoulder, from the heave and traffic.

After a winter with a large number of freeze-thaw cycles the granular material will be weakened and have a severe loss of support (14). This is shown in Figure 4.20 for resilient modulus and compressive strength (23, 24). Traffic will fatigue these materials at a rapid rate following a winter's freeze-thaw activity. This damage will be even greater when there is an accumulation of moisture.

In the western states the problems of frost heave will not be as great. The effect of temperature cycling and moisture may be more prevalent however. This clearly defines the necessity of including environmental effects in the analysis and design procedure for shoulders.

Even though a joint may function, and remove the surface moisture from the pavement to the edge of the shoulder, this will also cause problems at the edge of the shoulder; as this is where the moisture will collect and weaken the shoulder support. This is shown in a study by Russam and Dagg (25) who studied moisture problems at pavement edges. A portion of their work is shown in Figure 4.21. The concentration of moisture and moisture change is primarily at the outer edge of the shoulder. This concentration of moisture will produce a loss of strength in the shoulder material. When a vehicle parks on the shoulder, the outside edge of the shoulder section, commonly designed thinner than the pavement edge, will not be able to support the load. Severe rutting, or shoving, and corrugations will develop with a loss in structural integrity. With time this deterioration, along with the moisture concentration will progress closer to the pavement edge. This damage is common principally on asphaltic concrete shoulders where cracking under load will allow moisture into the granular material. The effect of moisture at

the outer edge of a concrete shoulder is less than it is for asphalt because of the increased stiffness and strength provided by the concrete.

4.4.6 EXPANSIVE CLAY

A final problem realted to moisture is the problem of expansive clays. This problem is similar to frost heave in that the shoulder and pavement will be thrust upward, producing forces similar to frost heave. The major difference is that expansive clays are influenced almost entirely by moisture. Proper drainage may be critical in controlling expansive clay, or it may serve no function since the expansion is often controlled by the water table.

The effect of expansive clays differs from frost heave in that the heave is not uniform along the length of the roadway and will vary in height and spacing at regular intervals (25). Because the overburden pressure of the shoulder is less than that of the pavement, the heave in the shoulder will be greater than the pavement. Some typical wave lengths (distances from peak to peak of the expansion) are shown in Figure 4.22. The wave length and magnitude have been related to the Present Serviceability Index (PSI) by Walker (27). The weighted amplitude in Figure 4.23 is the amplitude that is larger than 99 percent of all others, multiplied by the probability that its associated wave length will occur along a given length of road (28). Thus the value could come from a few very large bumps or a large number of very small bumps at regular intervals. The major point is that it does relate very well with the PSI, as shown in Figure 4.23.

The major consequence of the expansive clay roughness is to produce a dynamic load on the pavement. Because the magnitude of the roughness will be greater on the shoulder, the dynamic load will be greater, causing more fatigue damage from the larger stresses. The relationship between the

dynamic load as influenced by the PSI is given in Figure 4.24. Since the shoulder will typically have a lower PSI than the mainline pavement, dynamic loading might be quite severe on the under designed sections of shoulders and would greatly accelerate normal fatigue.

4.5 SUMMARY

The discussion in this chapter points out the problems that are commonly seen on shoulders throughout the United States. Although some shoulders have performed satisfactorily, they have not been designed from a rational procedure, on the whole. The majority of the problems for the shoulder are similar to those found in the mainline pavement, but they are multiplied by two major factors:

1) The shoulder is commonly a thinner structural section.
2) Moisture is concentrated at both edges of shoulders as constructed on today's highways.

The effect of moisture on the behavior of materials used in the design of shoulders clearly shows that the performance will deteriorate quite rapidly when the materials are exposed to moisture. Temperature variations in the shoulder are going to be different from those in the mainline pavement, and this will create different stress conditions between the two. The performance of the shoulder suffers the most by these temperature differences.

The design of the shoulder is much more complicated than that of the mainline pavement. The exact amount of traffic is not known, the arrangements of the structural components is much more complicated, and the exact moisture and temperature condition of the materials is not known. Very little, if anything can be said about the stress and strain levels in the shoulder with such a lack of necessary information. There have been extensive studies

conducted that show how material properties will vary with moisture and temperature. This information is only qualitatively useful when the exact moisture and temperature variations in the shoulder structure are not known. The first step in a design will have to be the determination of the variation in these two quantities over the life of the shoulder. Once these are known, the calculated stresses and strains will be able to predict the life of the shoulder.

The shoulder-pavement joint will be critical to the performance of the shoulder. This is where water enters the pavement; and is where the water should be removed or eliminated. At the present time neither alternative can be considered a sure thing. The technology of moisture movement has not been properly applied to drains, and the use of joint sealants has been poor. Improvement of the pavement shoulder joint will improve performance of the shoulder.

Table 4.1

One Inch Surface Thickness

State	Surface	Base	Thickness	Subbase	Thickness
Alabama	AC	AC	3"	Select Soil	4 1/2"
Florida	AC	SA	5"	Sand Clay	6"
North Carolina	ST or AC	AB	8"	ASB	4"
New York	AC or ST	AB	3"	ASB	17"

Table 4.2

One and One-half Inch Surface Thickness

State	Surface	Base	Thickness	Subbase	Thickness
Georgia	AC	CTB	6"	Select Borrow	
Illinois	AC	CTB LTB ATB	6 1/2"	ASB	4"
Michigan	AC	ATB	6 1/2-7 1/2	ASB	14"
Minnesota	AC	AB	3"	ASB	9"-11"

Table 4.3

Two Inch Surface Thickness

State	Surface	Base	Thickness	Subbase	Thickness
Kentucky	AC	AB	Variable		
Missouri	AC	ATB CTB	5"	ASB	5"-7"
North Dakota	AC	Emulsion	6"	LTS	
South Dakota	AC	ATB LTB	6"	AC	2"
Washington	AC	AB	3"	ASB	7"

Table 4.4

Three Inch Surface Thickness

State	Surface	Base	Thickness	Subbase	Thickness
California	AC	AB	6"	ASB	Variable
Connecticut	AC	SSB	6"	NS	6"-18"
Maine	AC	AB	9"	ASB	9"
Ohio	AC	ATB	5"-6"	ASB	6"
Utah	AC	AB	6"	ASB	8"
West Virginia	PM	AB	6"	ASB	6"
Wisconsin	AC	AB	6"	ASB	15"

Table 4.5

Four Inch Surface Thickness

State	Surface	Base	Thickness	Subbase	Thickness
Arizona	AC	AB	5"	ASB	4"-6"
North Dakota	AC	ATB	4"		
Pennsylvania	AC or ST	AB	6"	ASB	12"
Idaho	AC	AB	8.9"	ASB	2.4"

Table 4.6

Thicker Surface Thicknesses

State	Surface	Base	Thickness	Subbase	Thickness
Kansas		AB	4"	LTS	6"
Louisiana	8"-10"	AC	3 1/2"	LTS	
Texas	8"	ATB	4"	LTS	

AC = asphalt concrete, ST = surface treatment, PM = penetration macadam
SA = sand asphalt CTB = cement treated base, ATB = asphalt treated base
LTB = lime treated base, AB = aggregate base, ASB = aggregate subbase
SSB = salt stabilized base, LTS = lime treated subgrade.

Table 4.7

Summary of Portland Cement Concrete Shoulder Design (5).

STATE	TYPE	SLAB THICKNESS (inches)	BASE TYPE	BASE THICKNESS (inches)	TIE BARS SIZE NO.	TIE BARS SPACING (inch)
Alabama	CRC	8	Aggregate	6		
Arizona		Design Details Not Available				
Georgia	Plain	11 taper to 6	Subgrade		4	30
Idaho		Design Details Not Available				
Illinois	Plain	6 min.	Subgrade		4	30
Iowa	Plain	6				
Kentucky	Plain, rein.	5 to 7				
Maryland	Reinforced	7			4	30
Michigan	Plain	9 taper to 6-1/4	Aggregate	4	hook bolt	40
Minnesota		Design Details Not Available				
Nebraska	Plain	5-1/2	Subgrade			
New Mexico	Plain	8	Cement Stab. Base			
New York	Plain	6 min.	Aggregate	8" min.		
N. Carolina	Plain	7				
N. Dakota	CRC	8	Aggregate	2	5	48
Pennsylvania	Plain	6	Aggregate	12	hook bolt	
Ohio						
Texas	CRC	8	C. Stab. Base	6	4	36
Utah	Plain	9	C. Stab. Aggre.	5	5	36
W. Virginia	Plain	8	C. Stab. Aggre.	6		

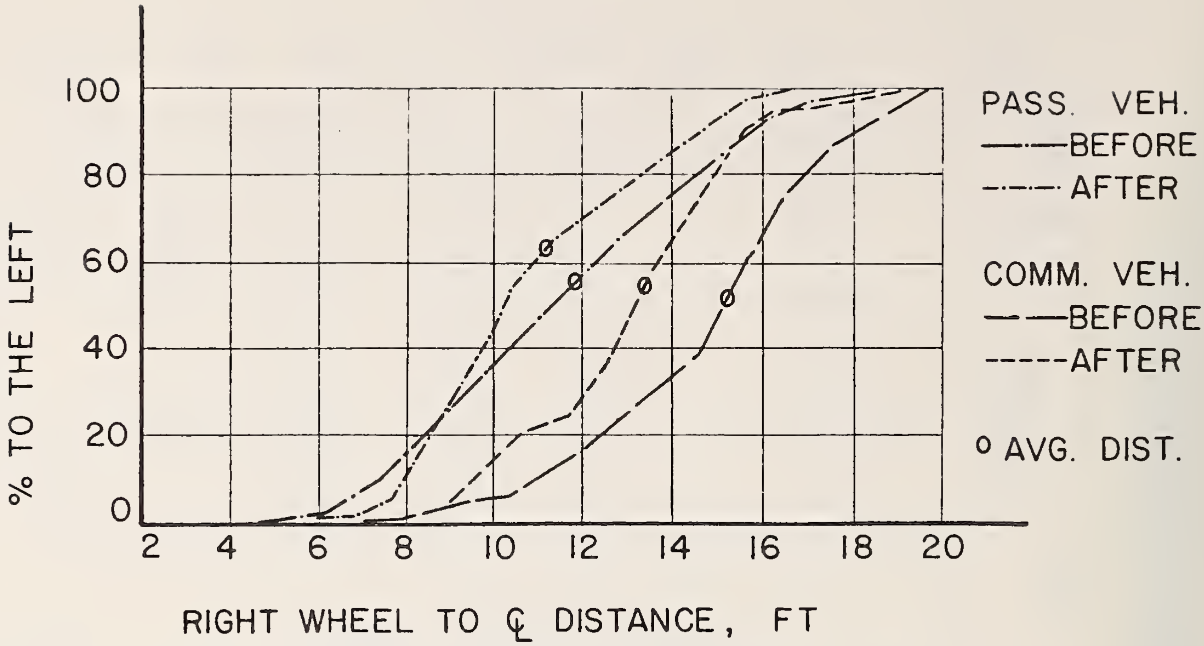


Figure 4.1. Position of Free Moving Vehicles Before and After Installation of Shoulder Stripes on 12 Foot Travel Lanes with 8 Foot Shoulders with No Contrast. Vertical Axis Represents the Percentage of Vehicles to the Left of the Distance Shown (3).

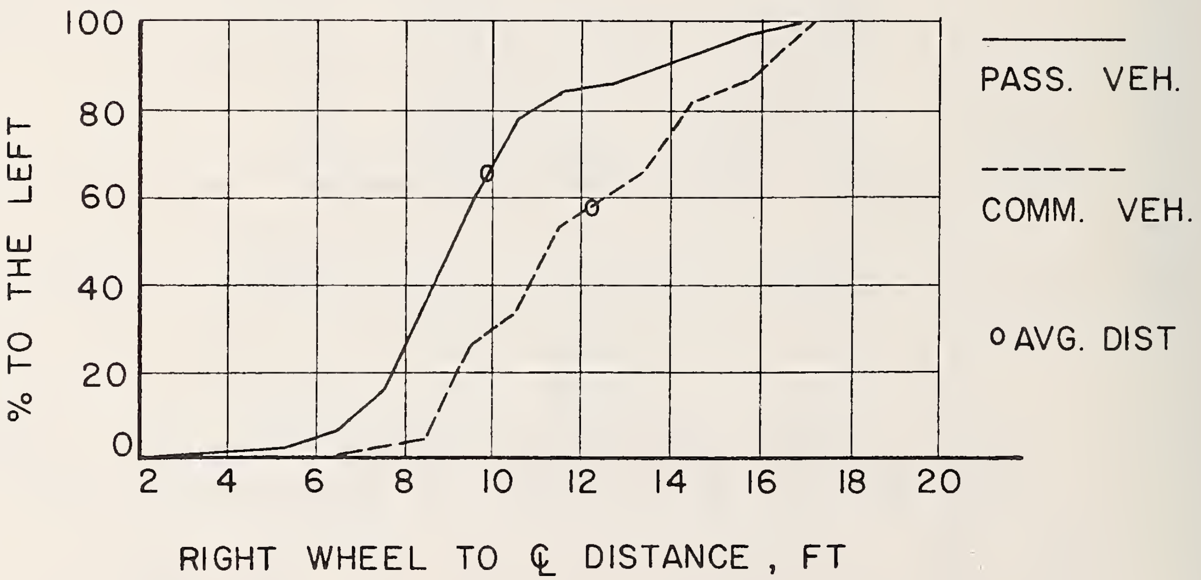


Figure 4.2. Position of Free Moving Vehicles on 12 Foot Travel Lanes and 8 Foot Shoulders wth Contrast. Horizontal Axis Represents the Distance from the Center of the Right Wheel to the Centerline of the Roadway (3).

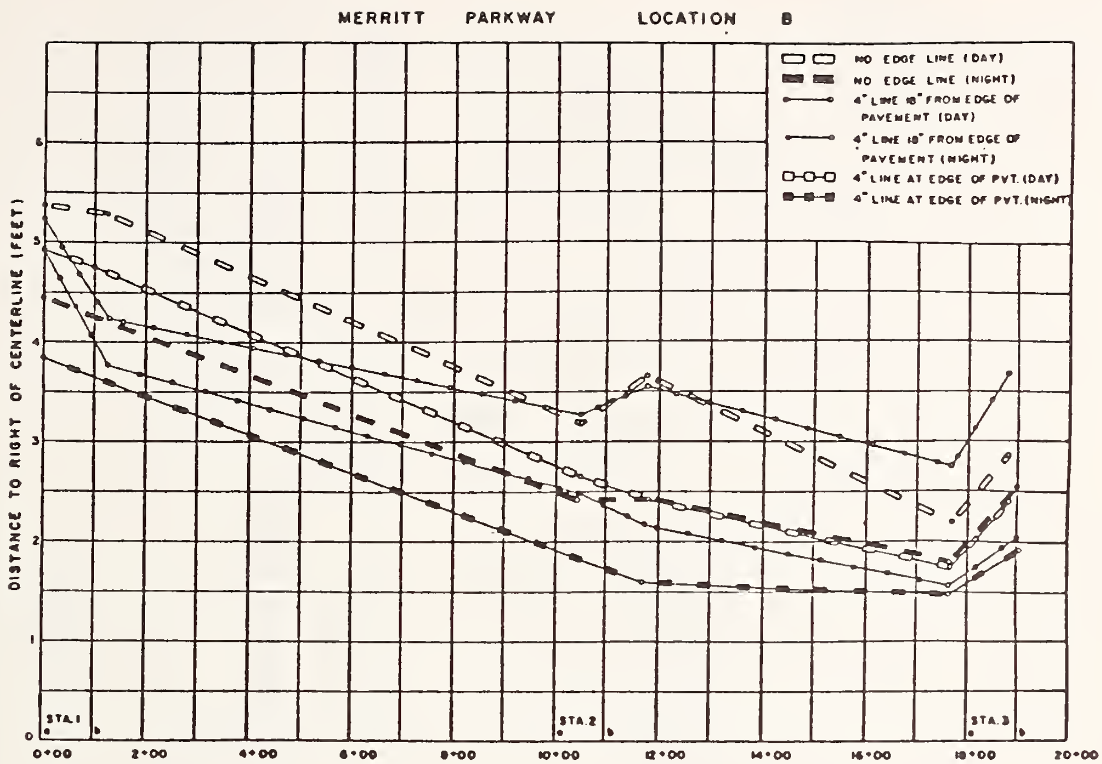


Figure 4.3. Effect of Edge Stripe on Vehicle Placement as Measured from Centerline to Left Wheel. Eighteen Inch Placement of Edge Stripe is Into the Travel Lane (6).

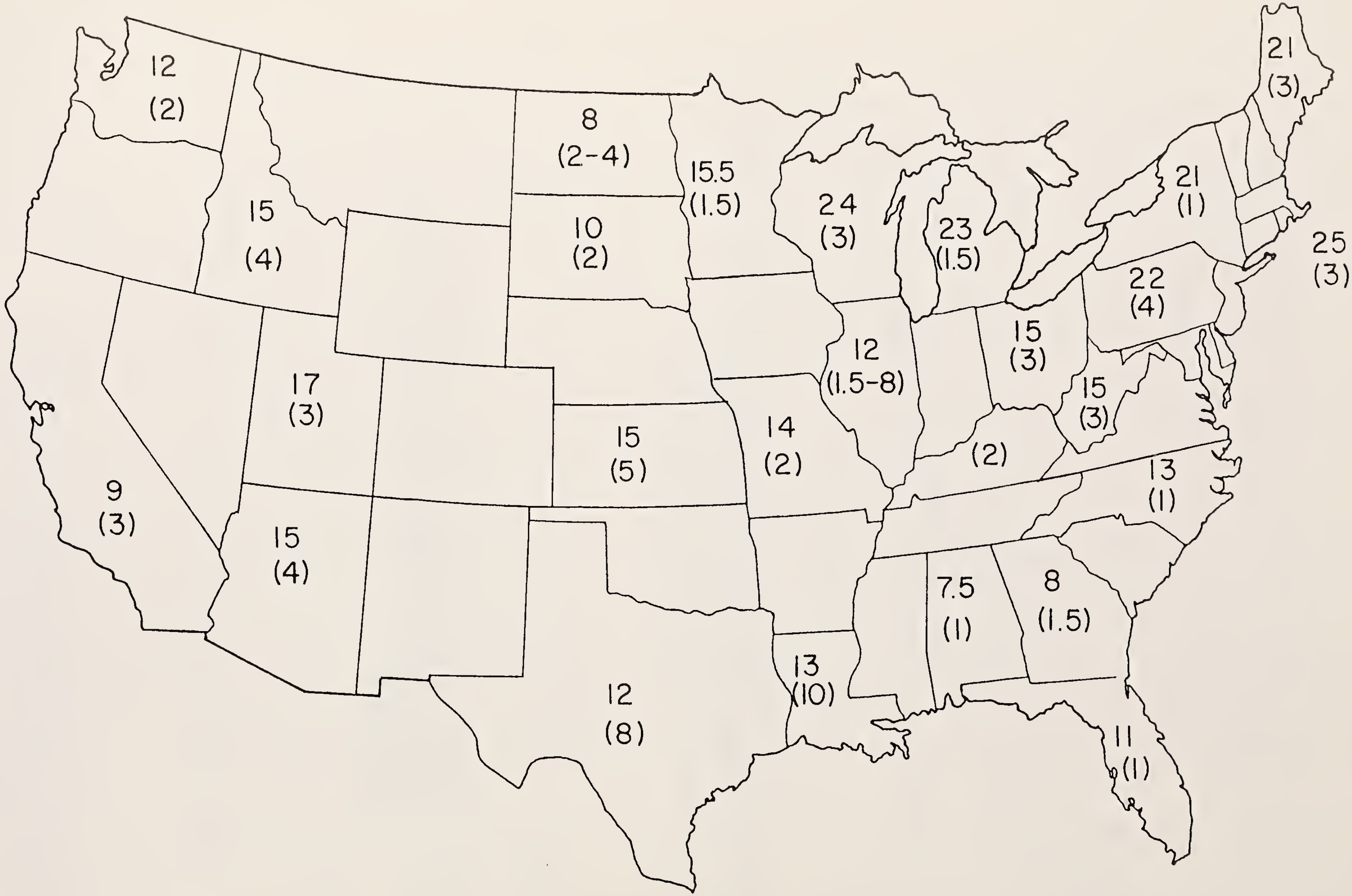


Figure 4.4. Regional Variation in Shoulder Thicknesses with Asphaltic Surfaces Indicated in Parenthesis.

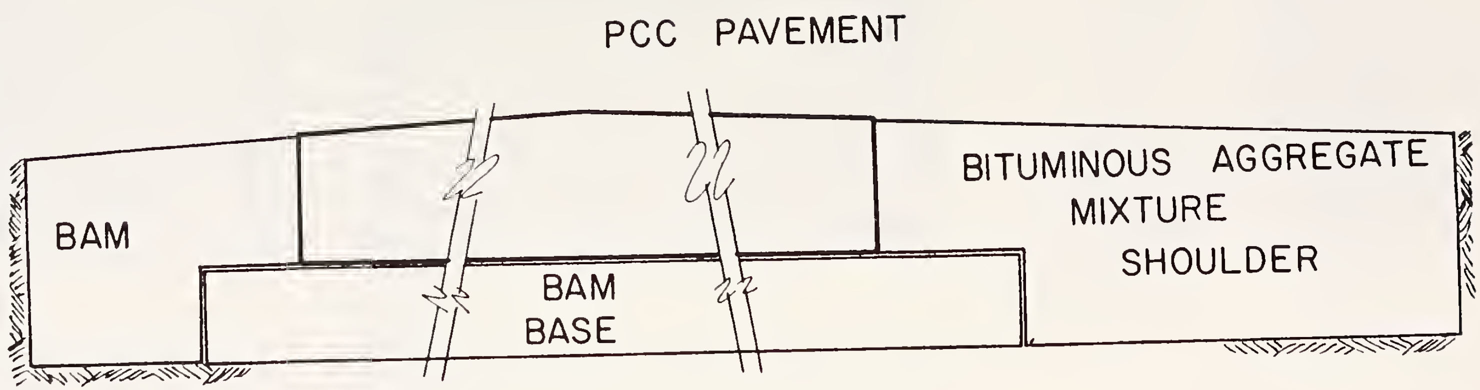
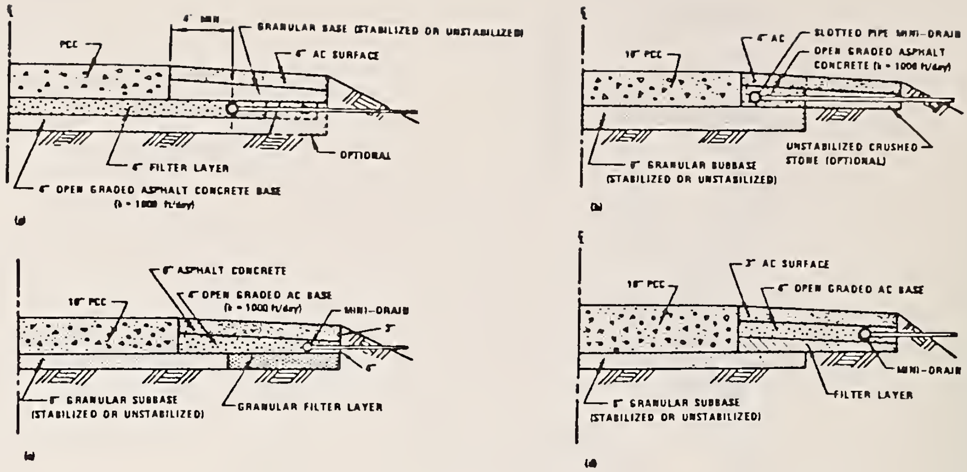
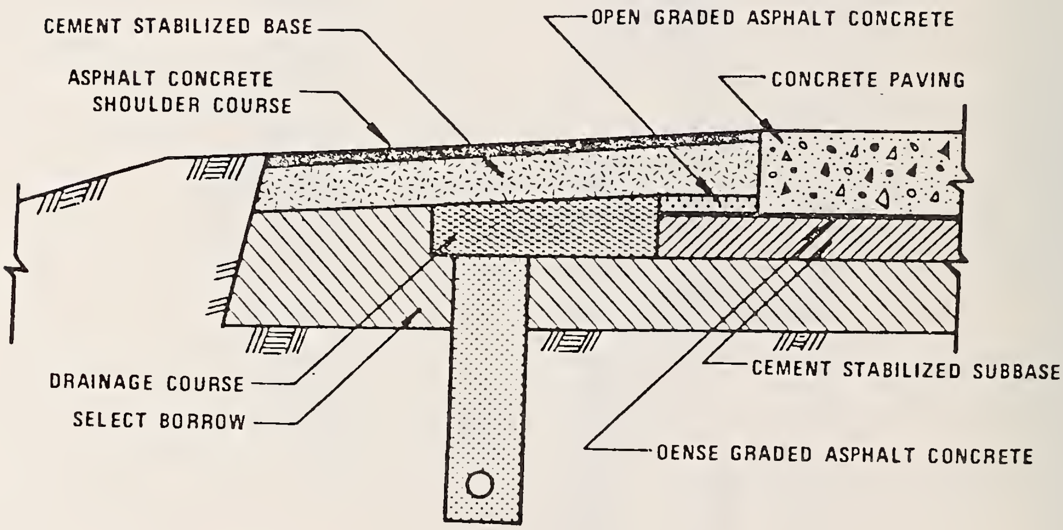


Figure 4.5. Common Illinois Cross Section with Shoulder Construction Resulting in Bathtub Type Design that Retains Water in Contact with Pavement.



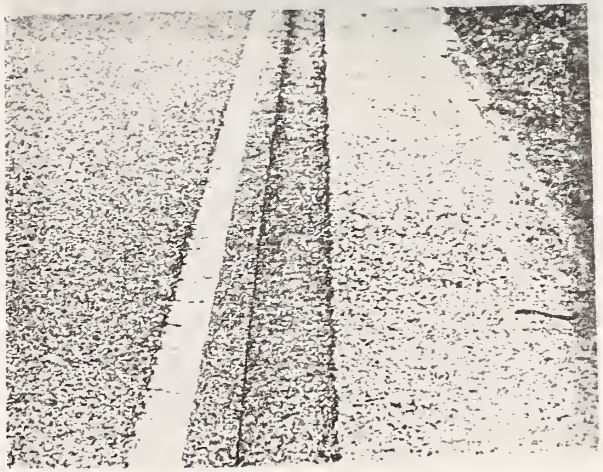
(a)



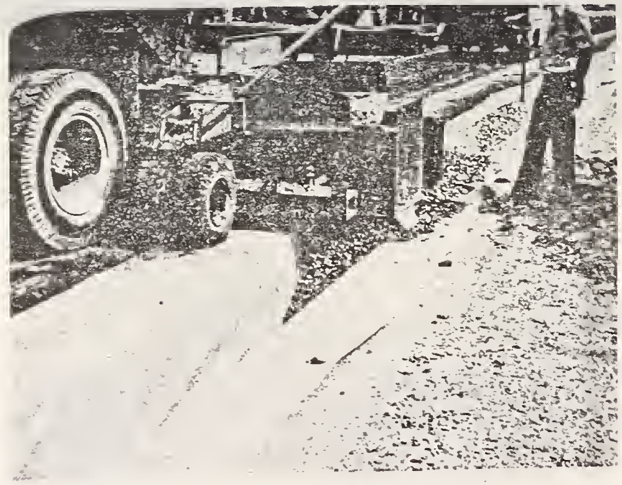
IMPROVED PAVEMENT-SHOULDER DRAINAGE SYSTEM USED IN GEORGIA

(b)

Figure 4.6 . Improved Shoulder Sections Using Drainage (5).

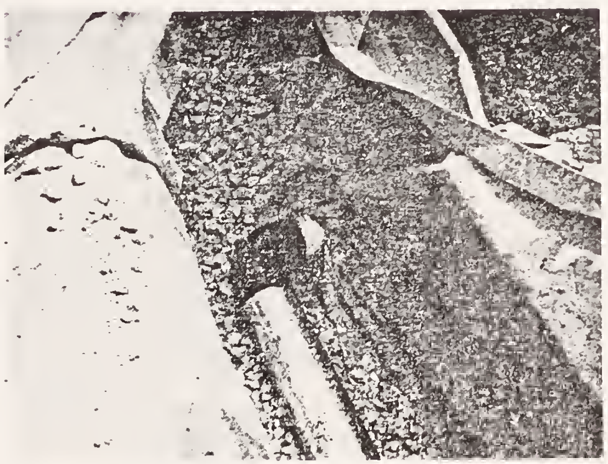


First pass of pavement cutter. This cut was then used to guide a double pavement cutter for removable of the pavement in the trench section.



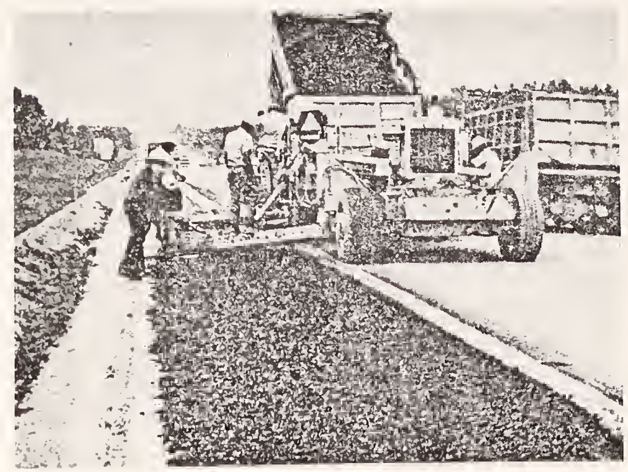
Filling trench with aggregate.

Finished trench. Notice the good line and grade.



Pipe outlet through the shoulder.

Finished trench ready for the pavement.



Wedge of plant mix ready for rolling.

Figure 4.7. Construction of Edge Drains in Alabama (13).

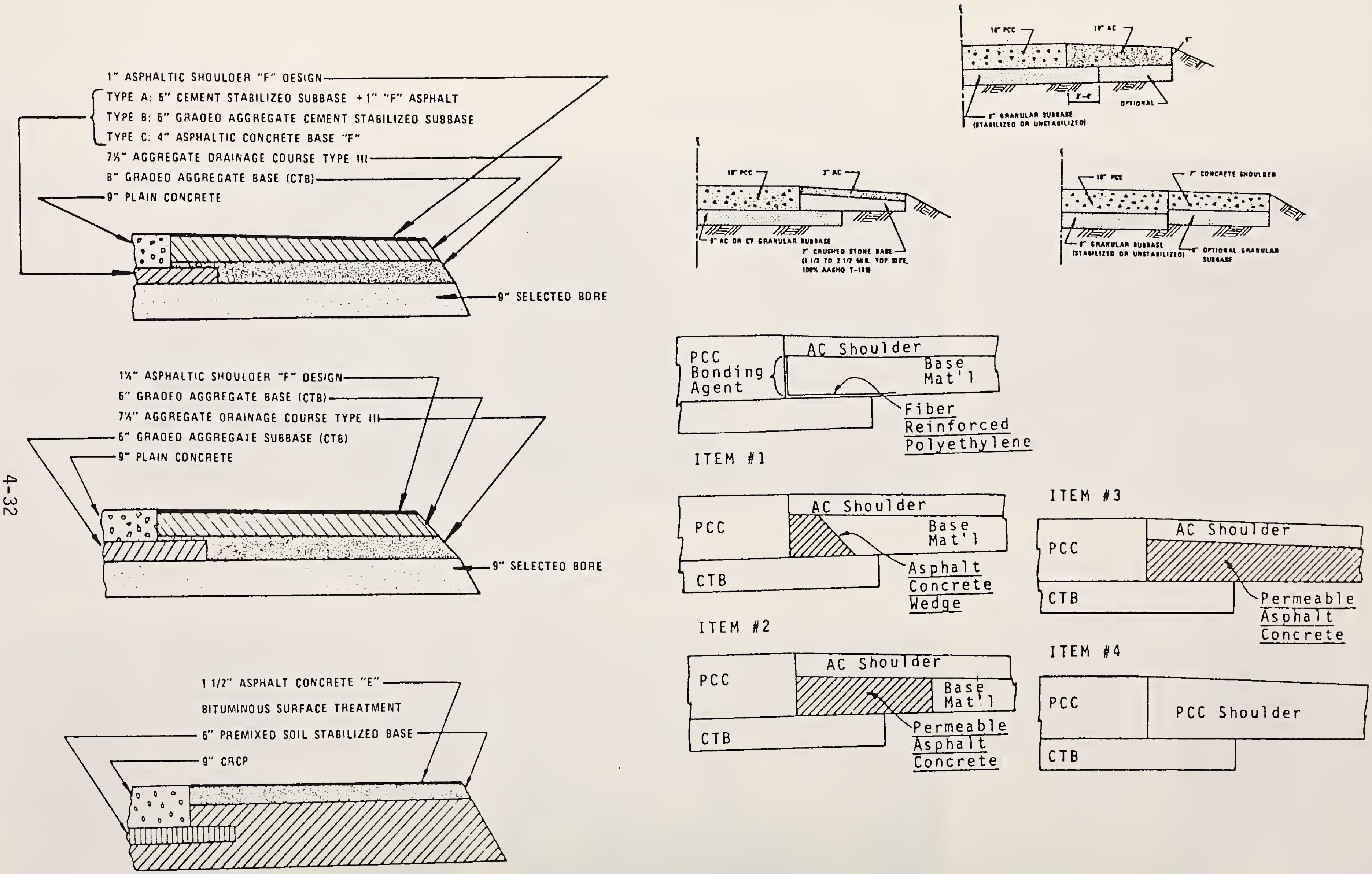


Figure 4. 8. Common Shoulder Sections (5, 12).

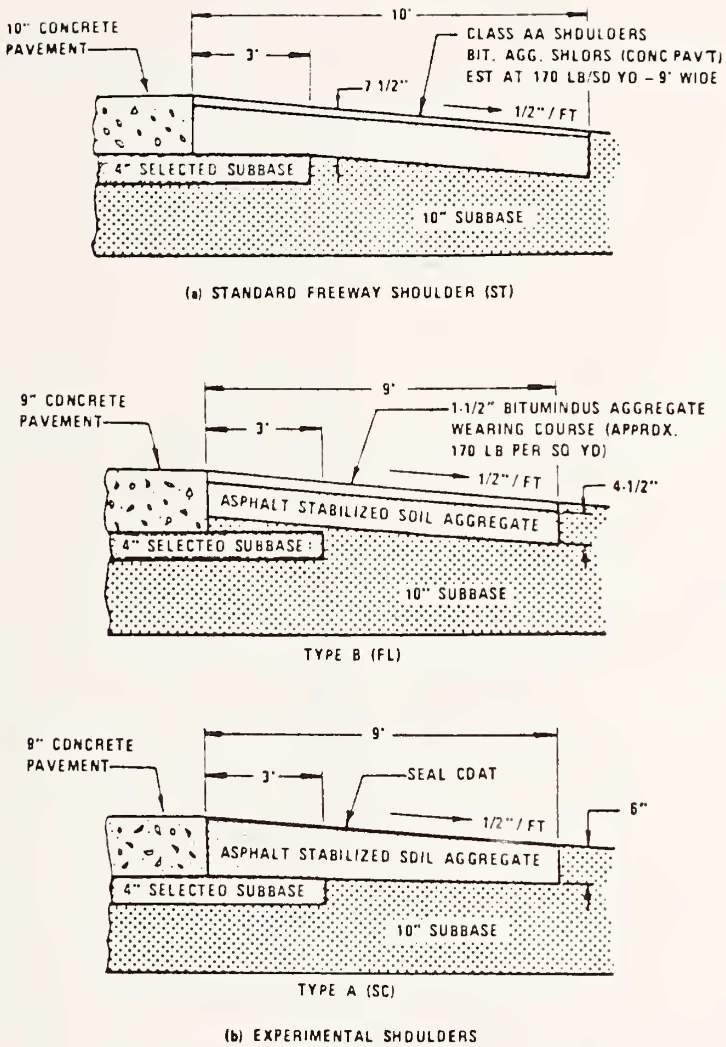


Figure 4. 9. Common Shoulder Sections in Michigan (5, 8).

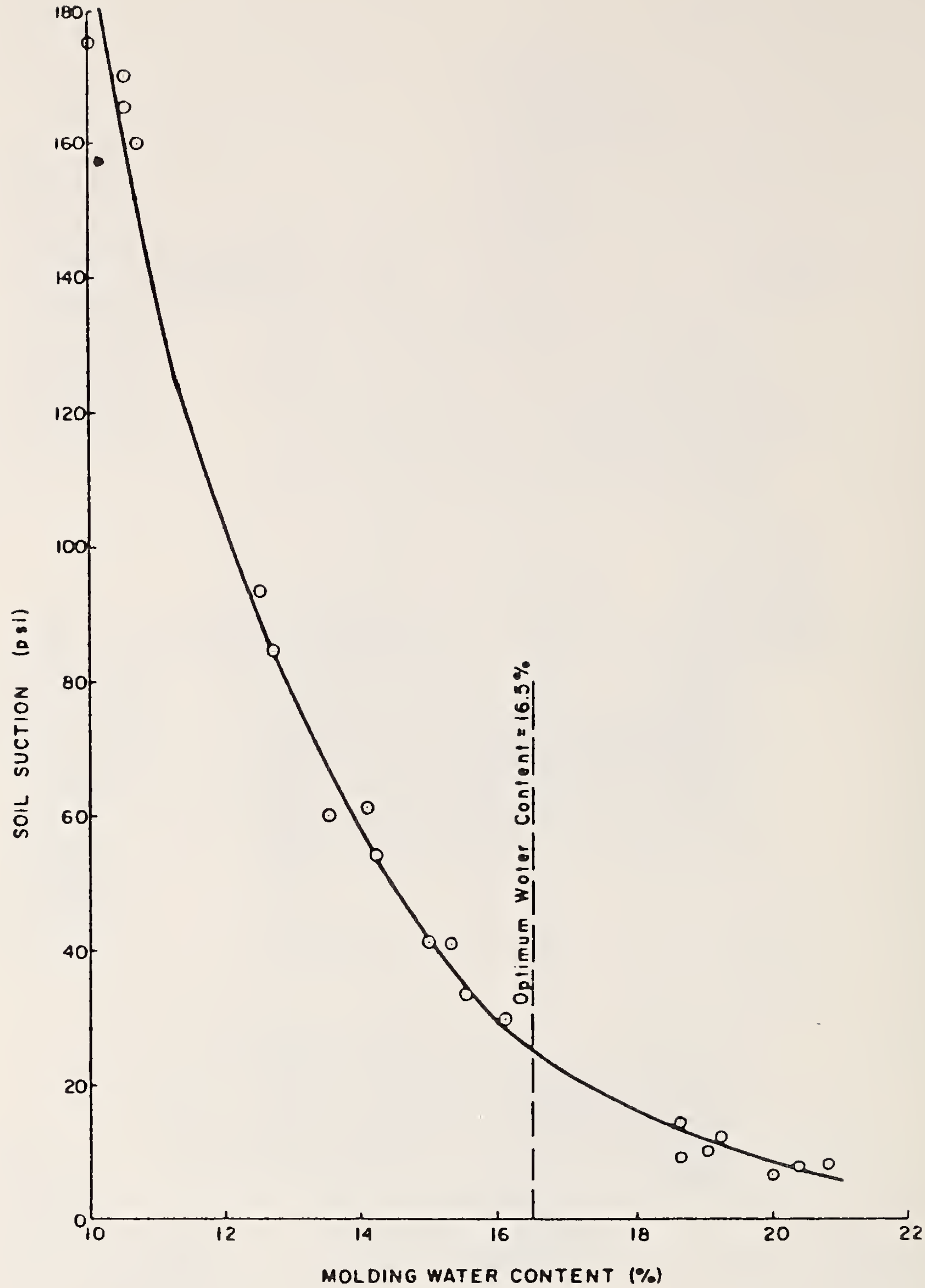


Figure 4.10. Soil Suction, Moisture Content Relationship for Laboratory Samples of Glacial Till (17).

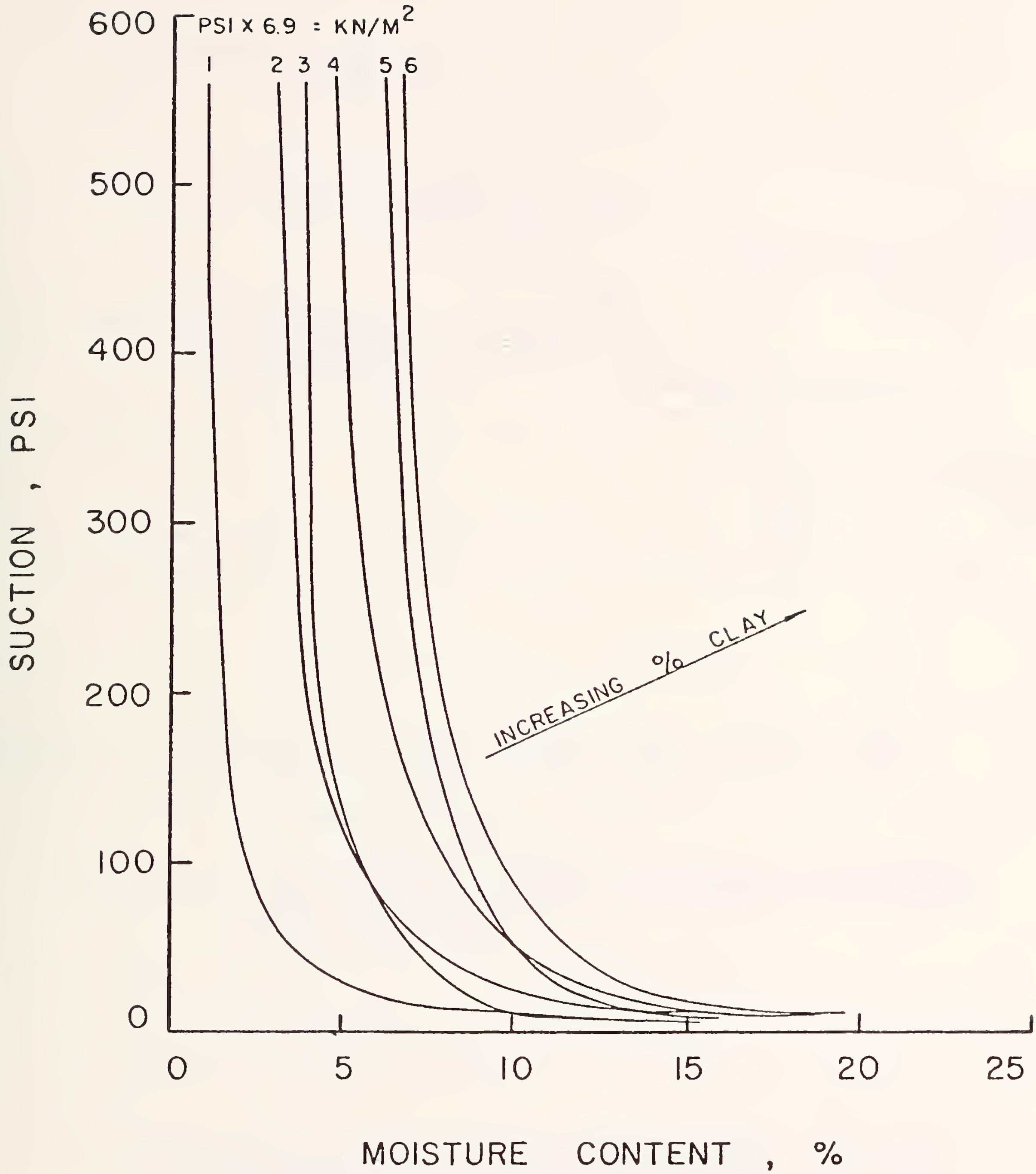


Figure 4.11. Suction as a Function of Moisture Content for Base Course Samples (20).

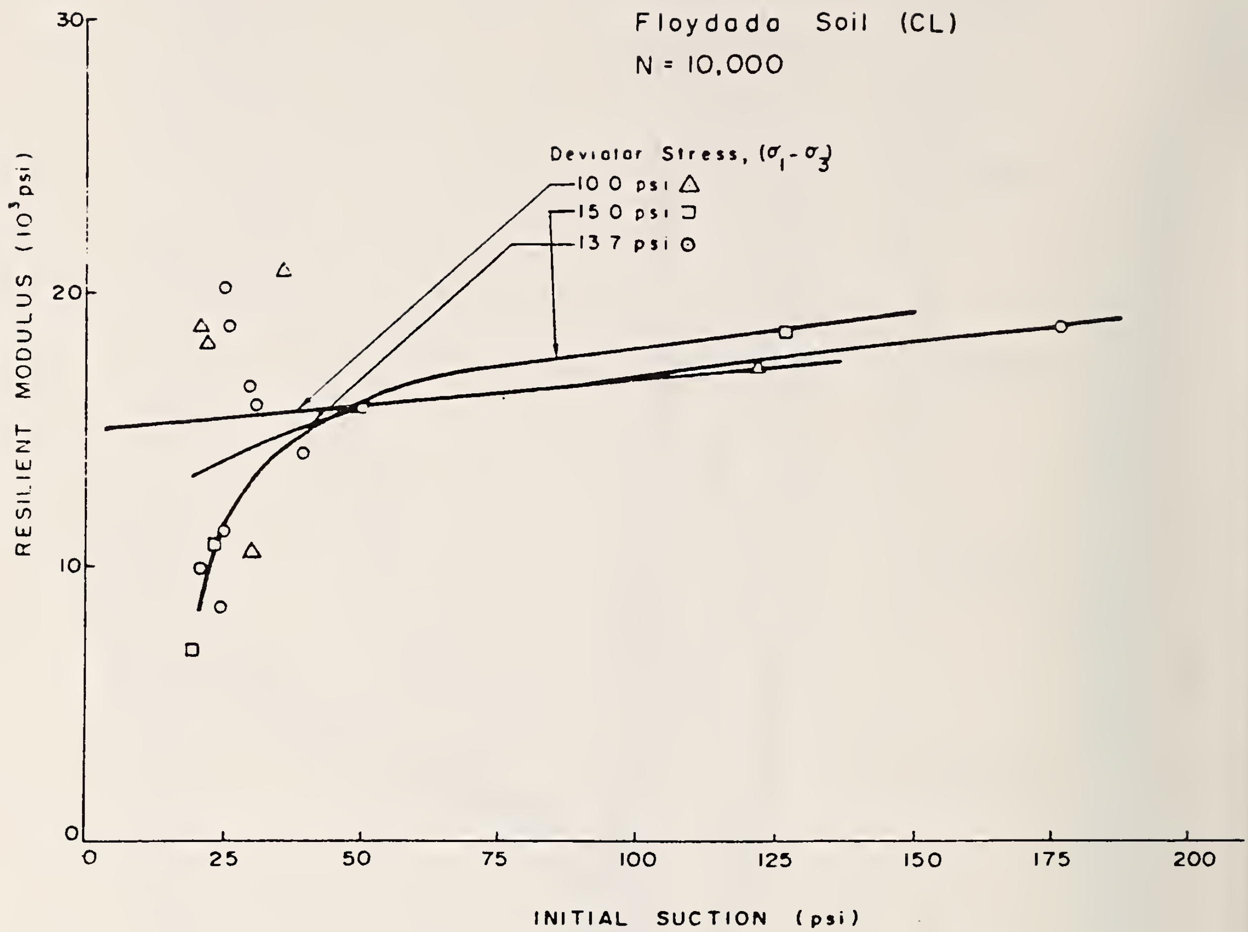


Figure 4.12. Resilient Modulus as a Function of the Initial Soil Suction at 10,000 Load Cycles (18).

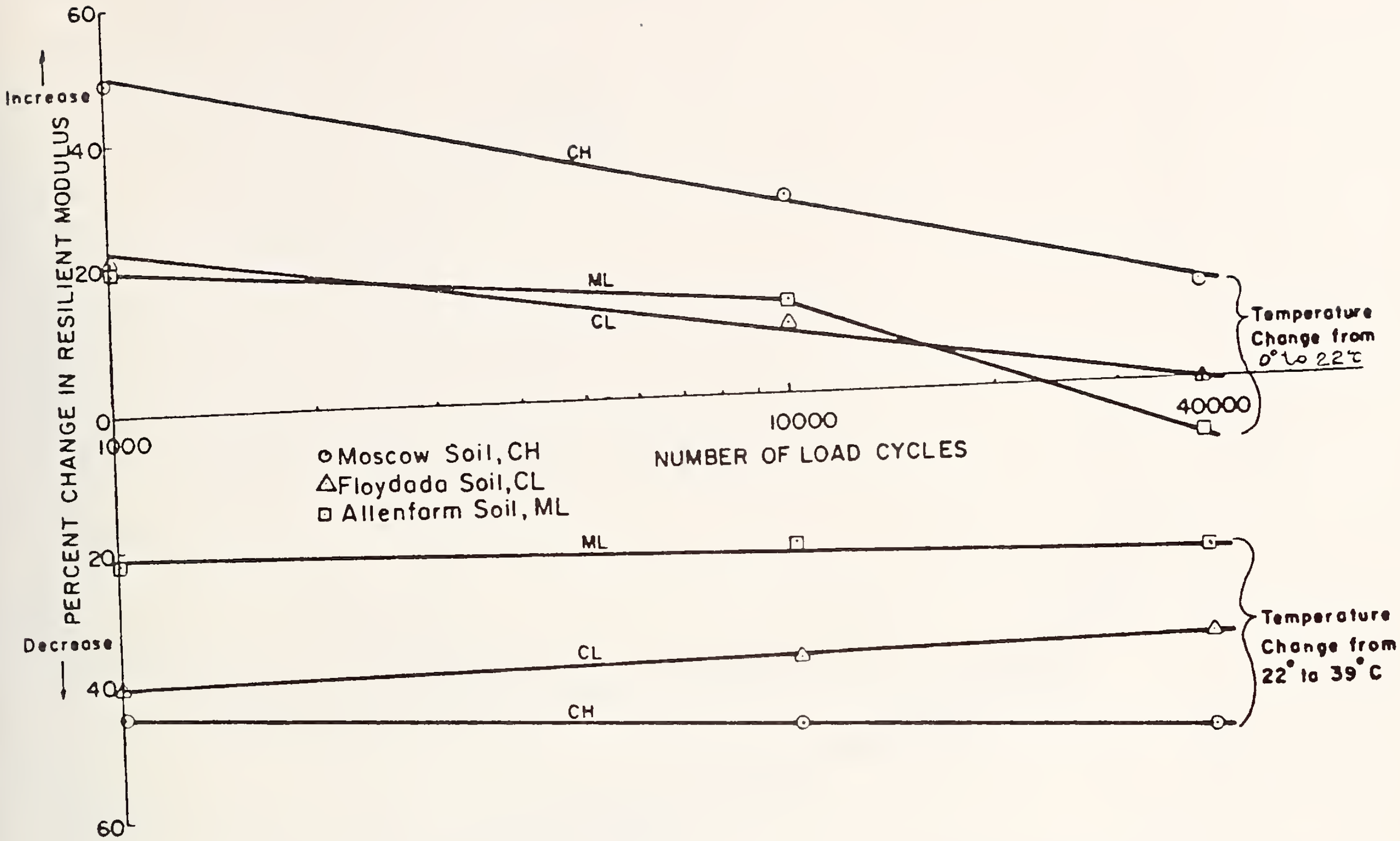


Figure 4.13. Percent Change in the Resilient Modulus of the Soils Due to Temperature Change as a Function of the Number of Load Cycles (19).

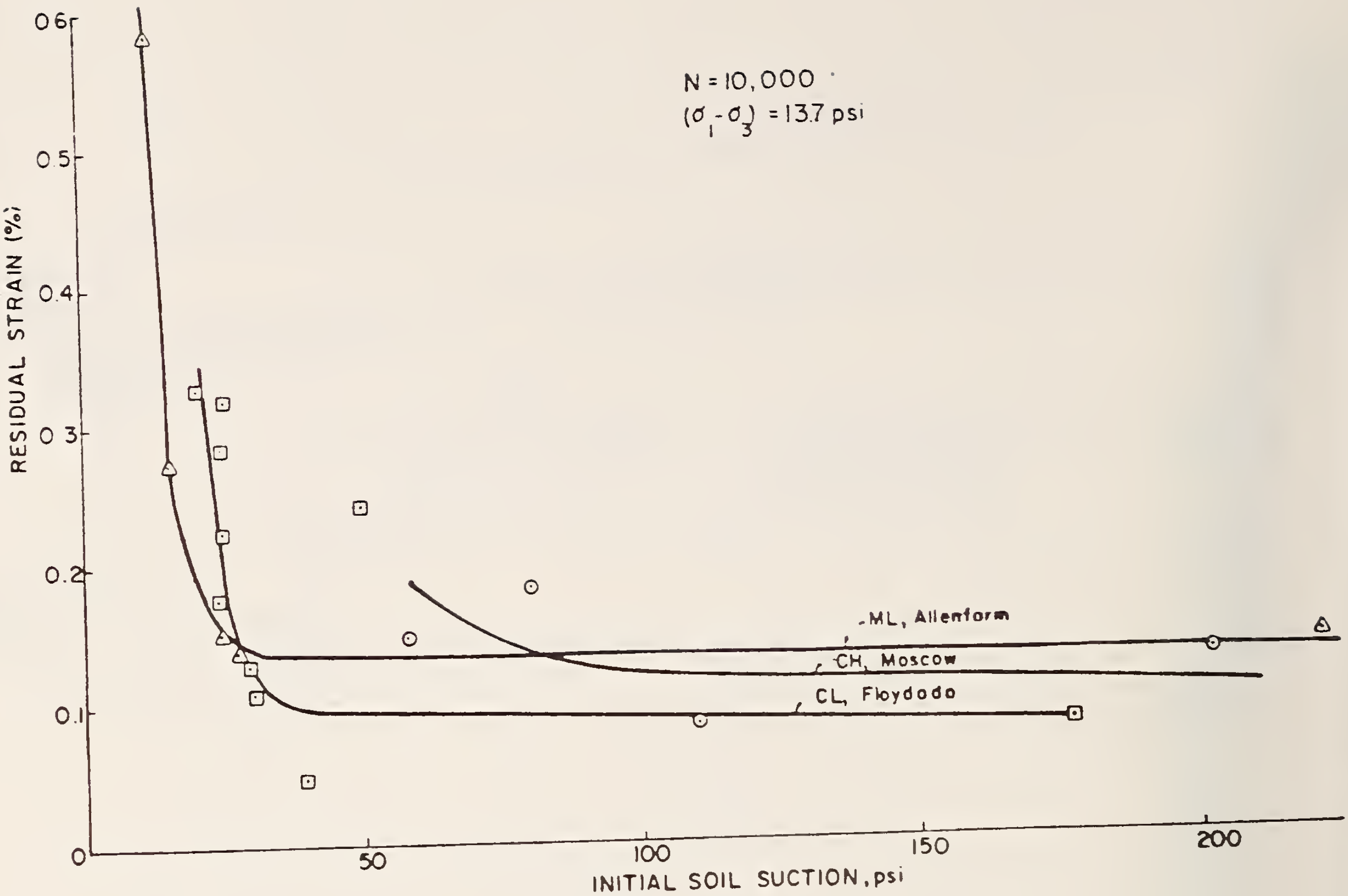


Figure 4.14. Residual Strain of Soils at 10,000 Load Cycles as a Function of the Initial Soil Suction (18).

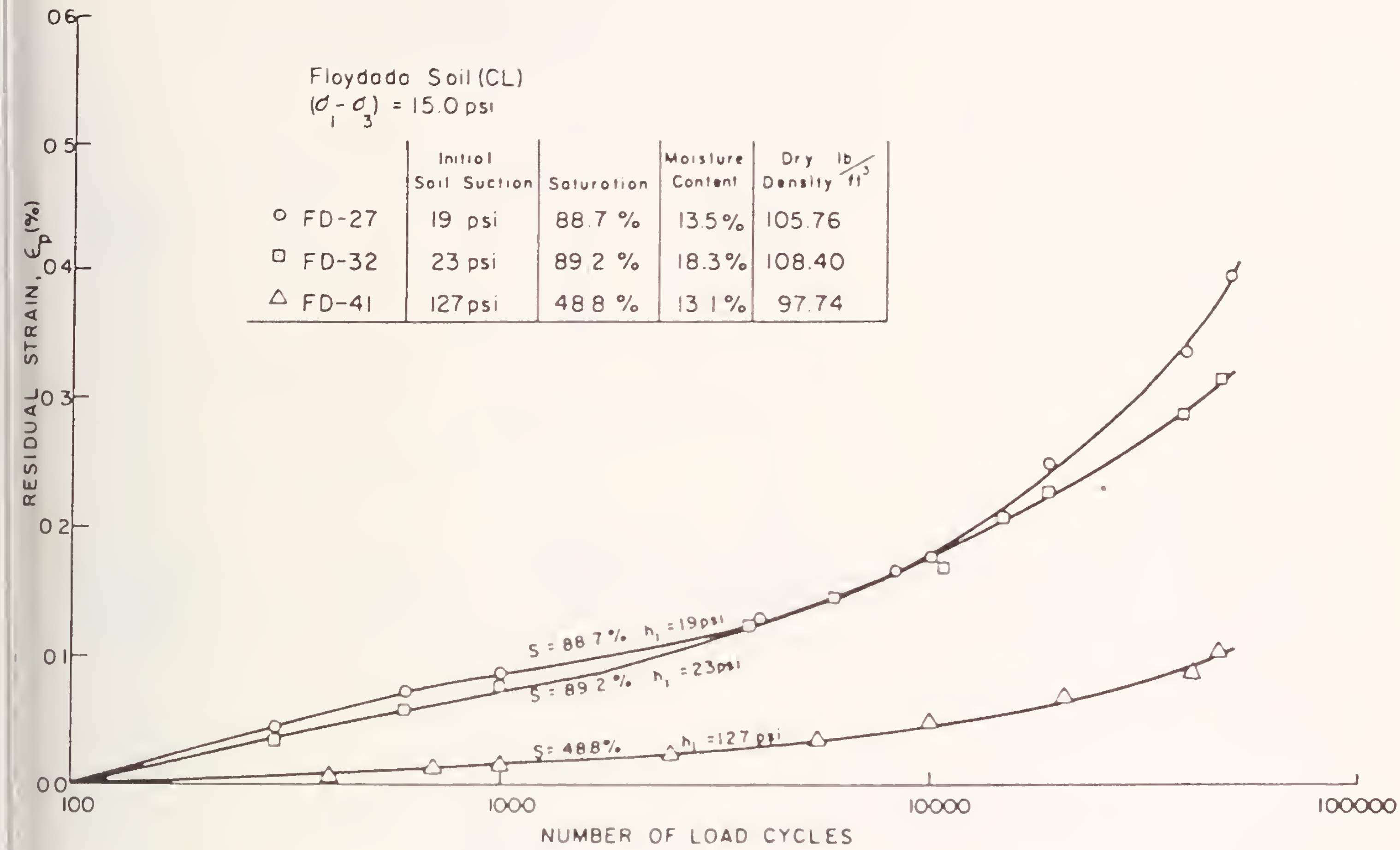


Figure 4.15. Variation of the Residual Strain with the Number of Load Cycles (18).

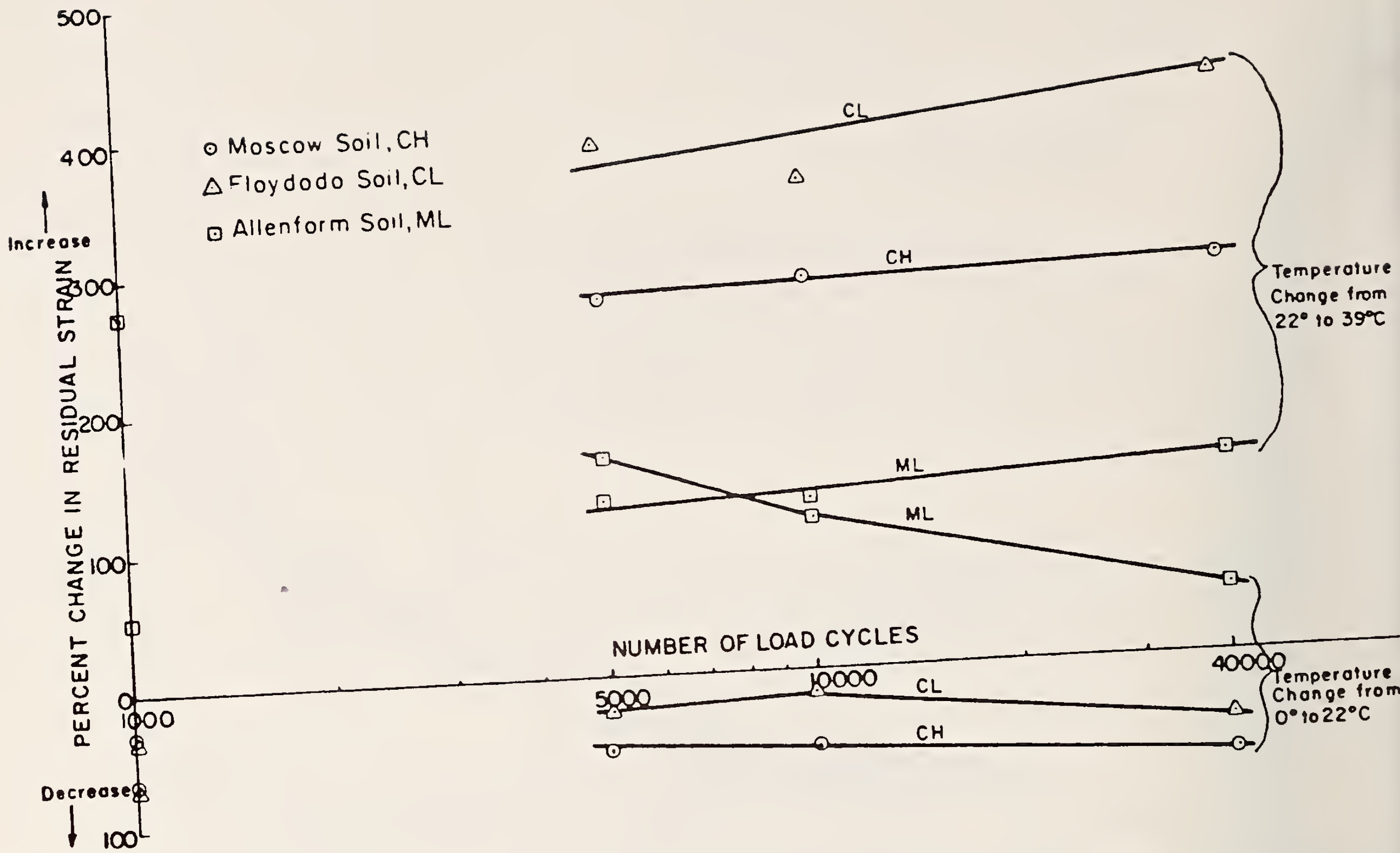


Figure 4.16. Percent Change in the Residual Strain of the Three Soils Due to Temperature Change as a Function of the Number of Load Cycles (18).

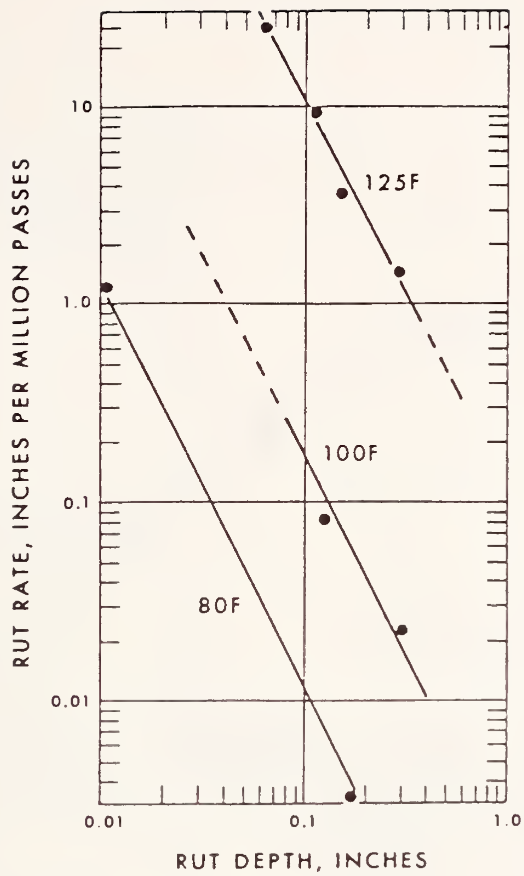


Figure 4.17. Dependence of Rut Depth on Rate of Rutting and Temperature (20).

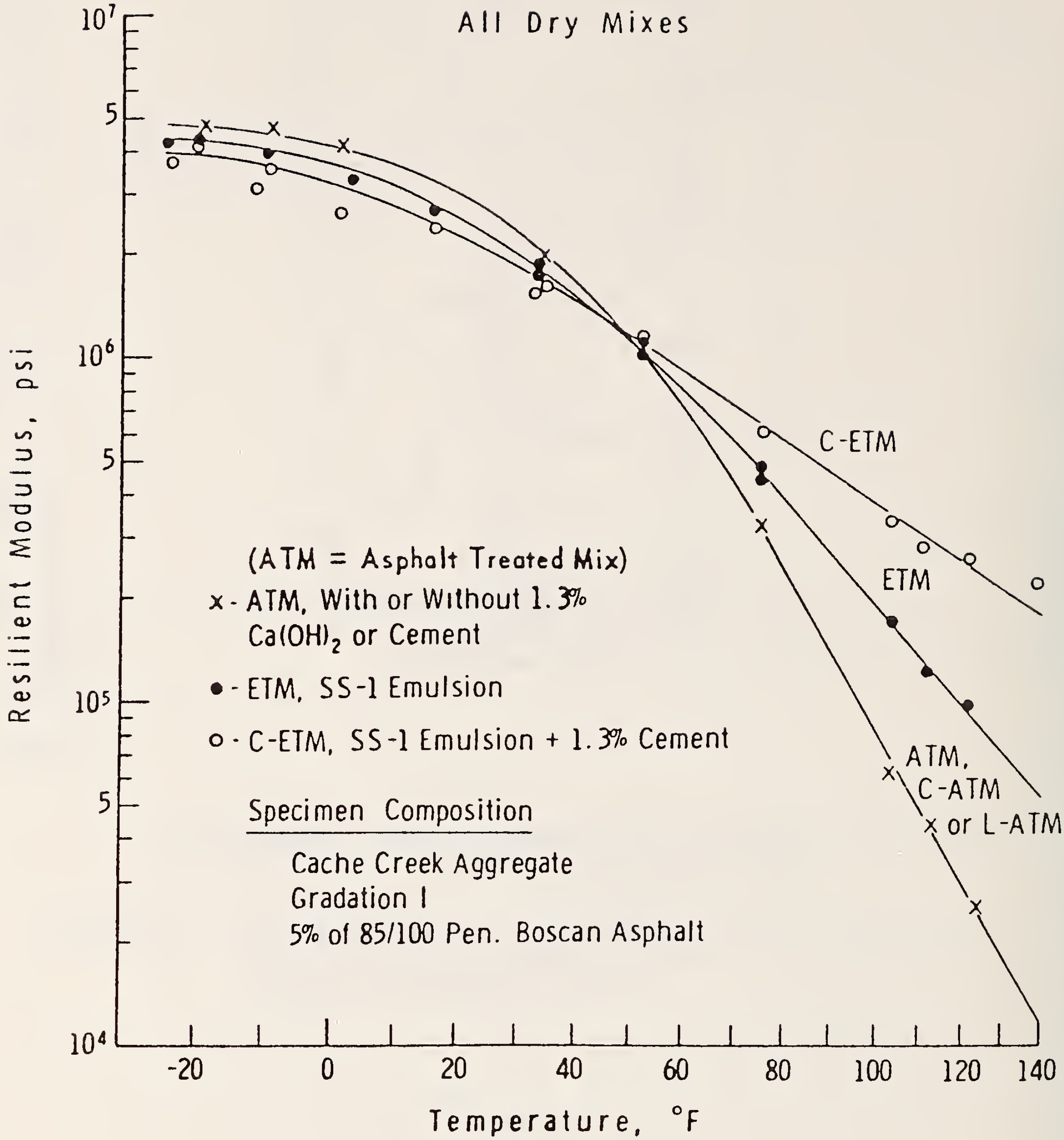


Figure 4.18. Effect of Temperature on the Resilient Modulus of Asphalt Treated Mixes (21).

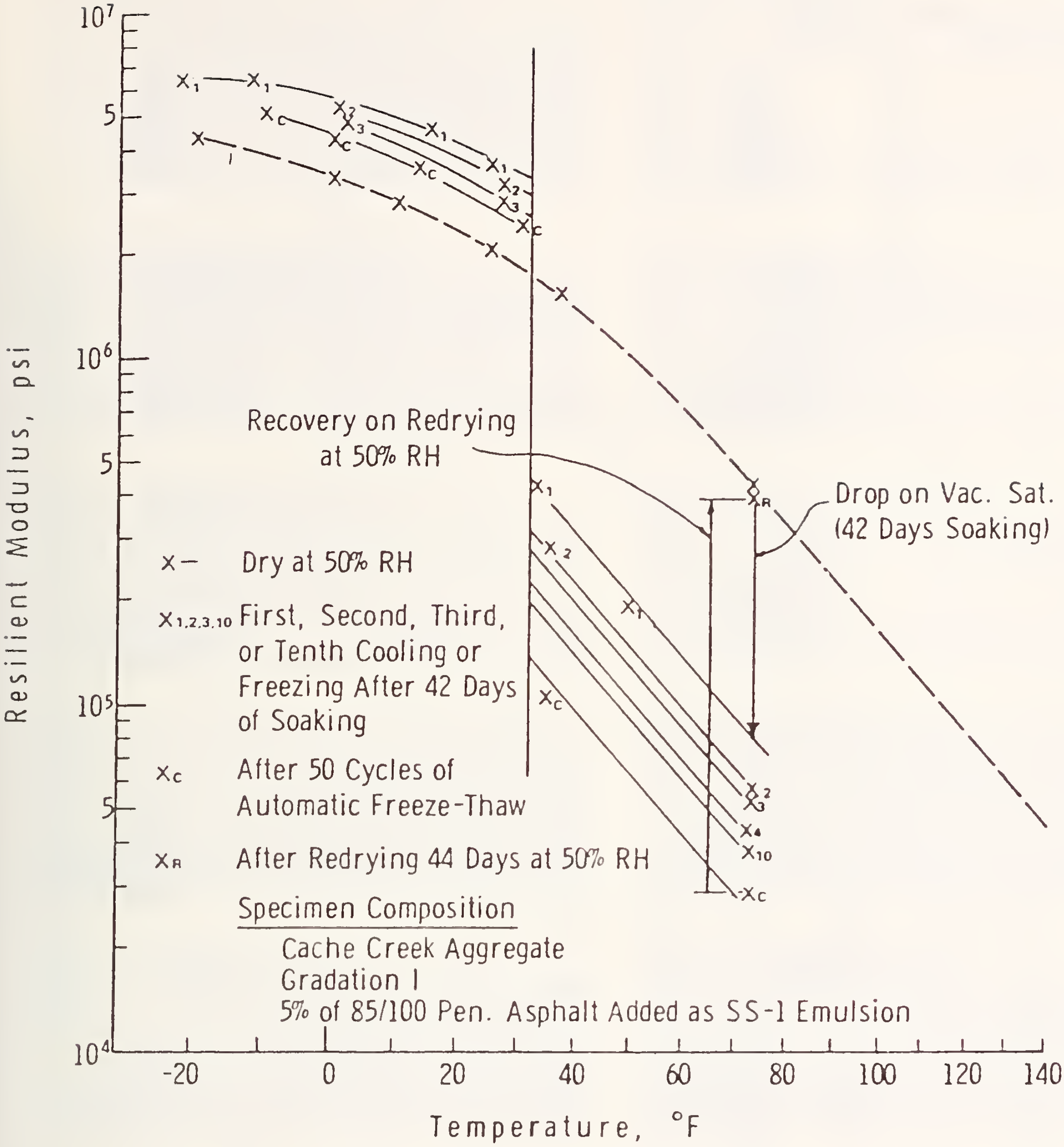
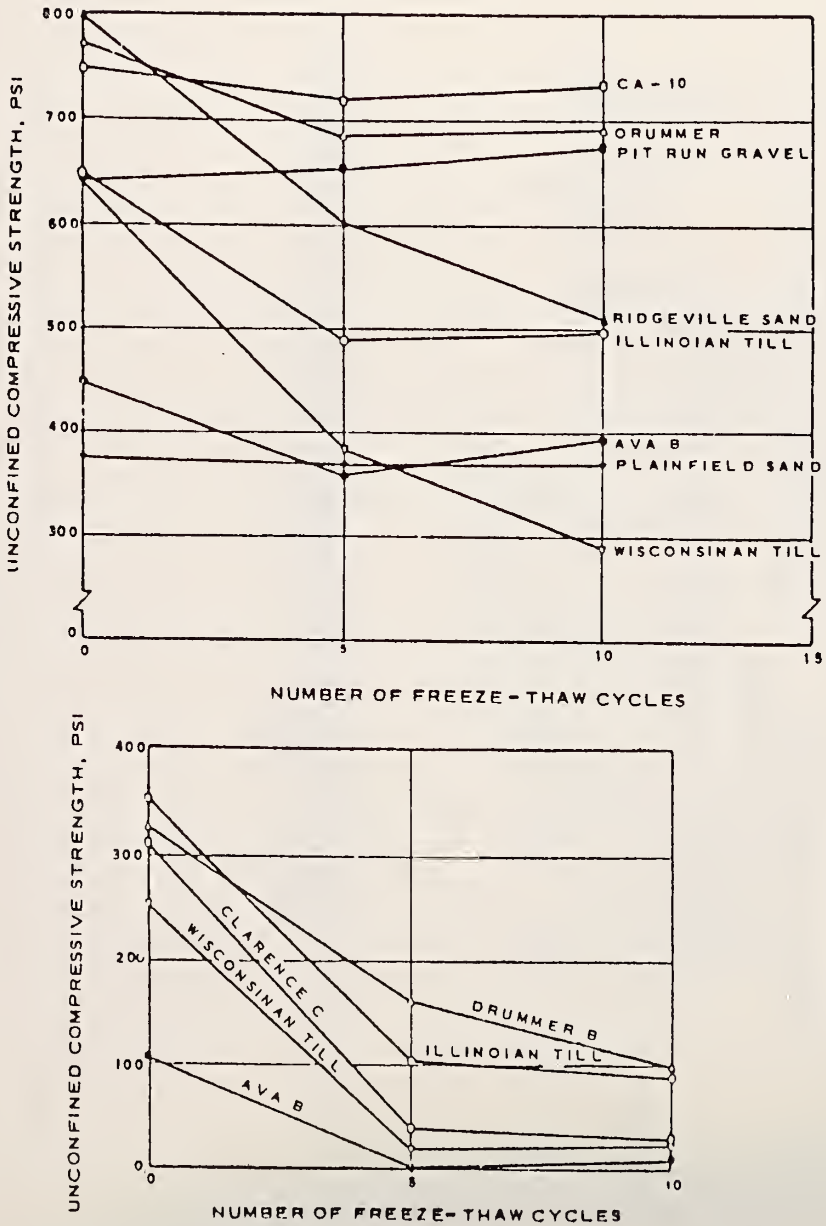
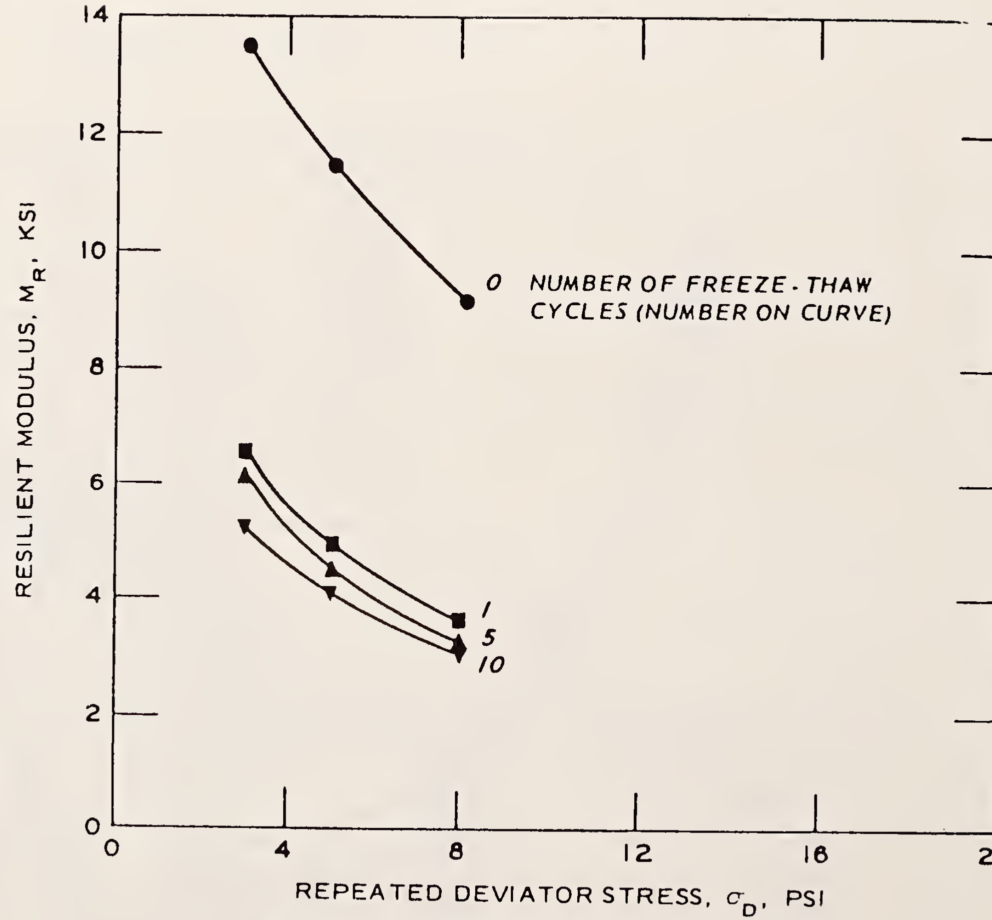


Figure 4.19. Freeze-Thaw Behavior of Asphalt-Emulsion-Treated Mixes, Showing Effects of Drying and Wetting on Resilient Behavior (21).



Effect of freeze-thaw on compressive strength (24).



Effect of freeze-thaw on resilient behavior of a clayey soil (23).

Figure 4.20. Weakening Effects of Winters' Activity.

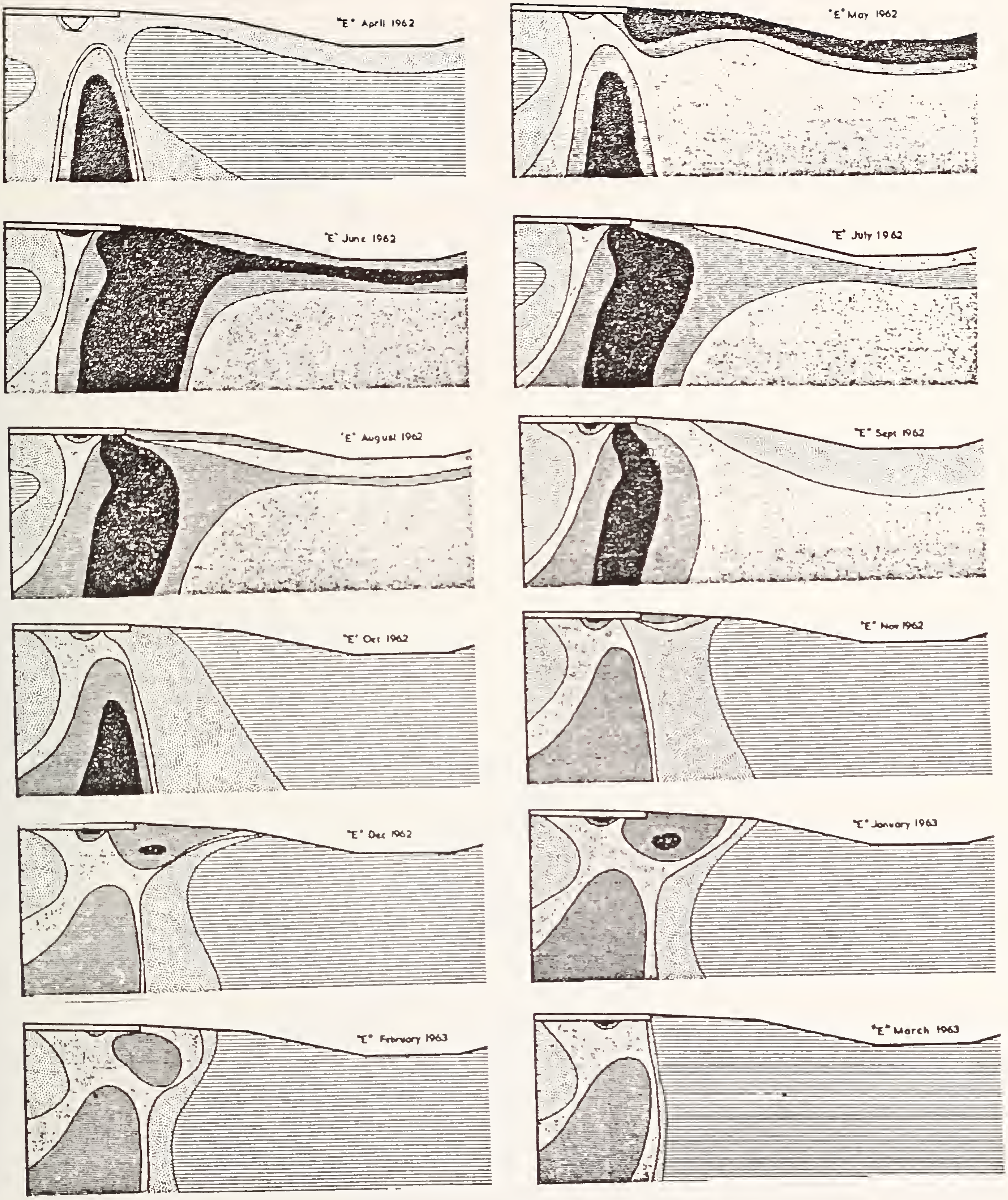


Figure 4.21. Change in Moisture Condition of Road Cross Section with Time at Site "E", Muguga (25).

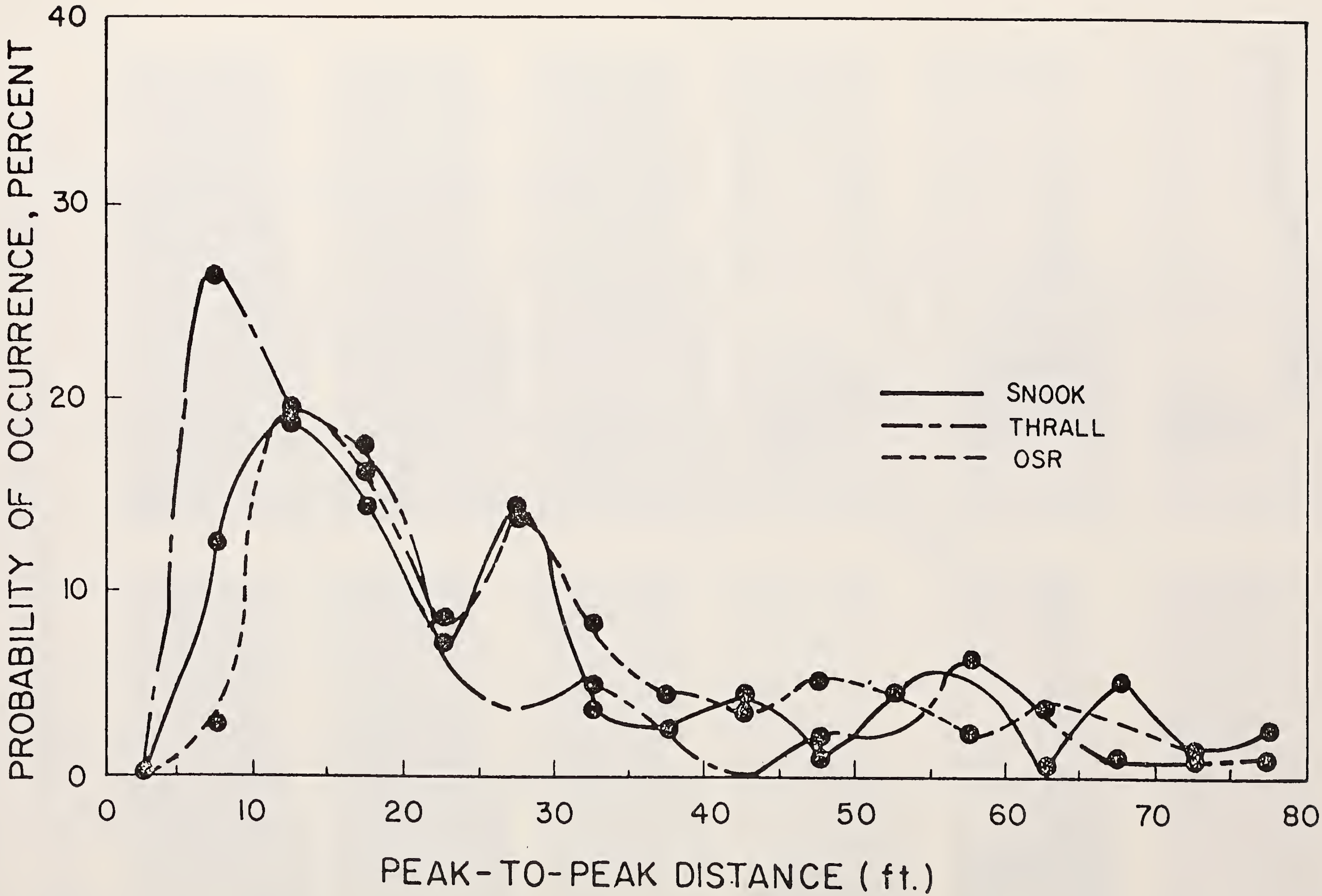


Figure 4.22. Probability Density Functions of Peak-to-Peak Distances, Left Wheel Path (26).

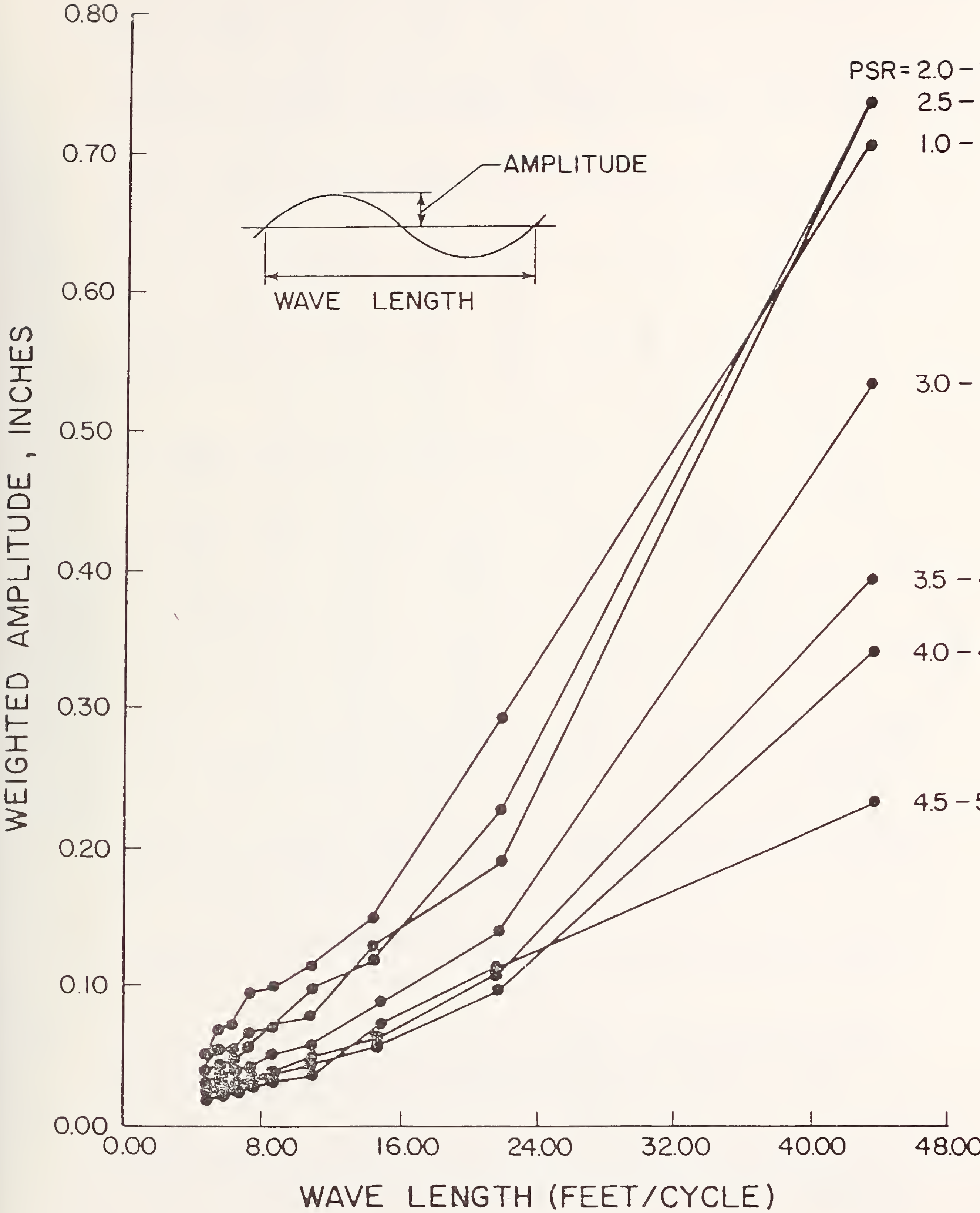


Figure 4.23. Amplitude Wave Length, and Serviceability Rating Relations (27).

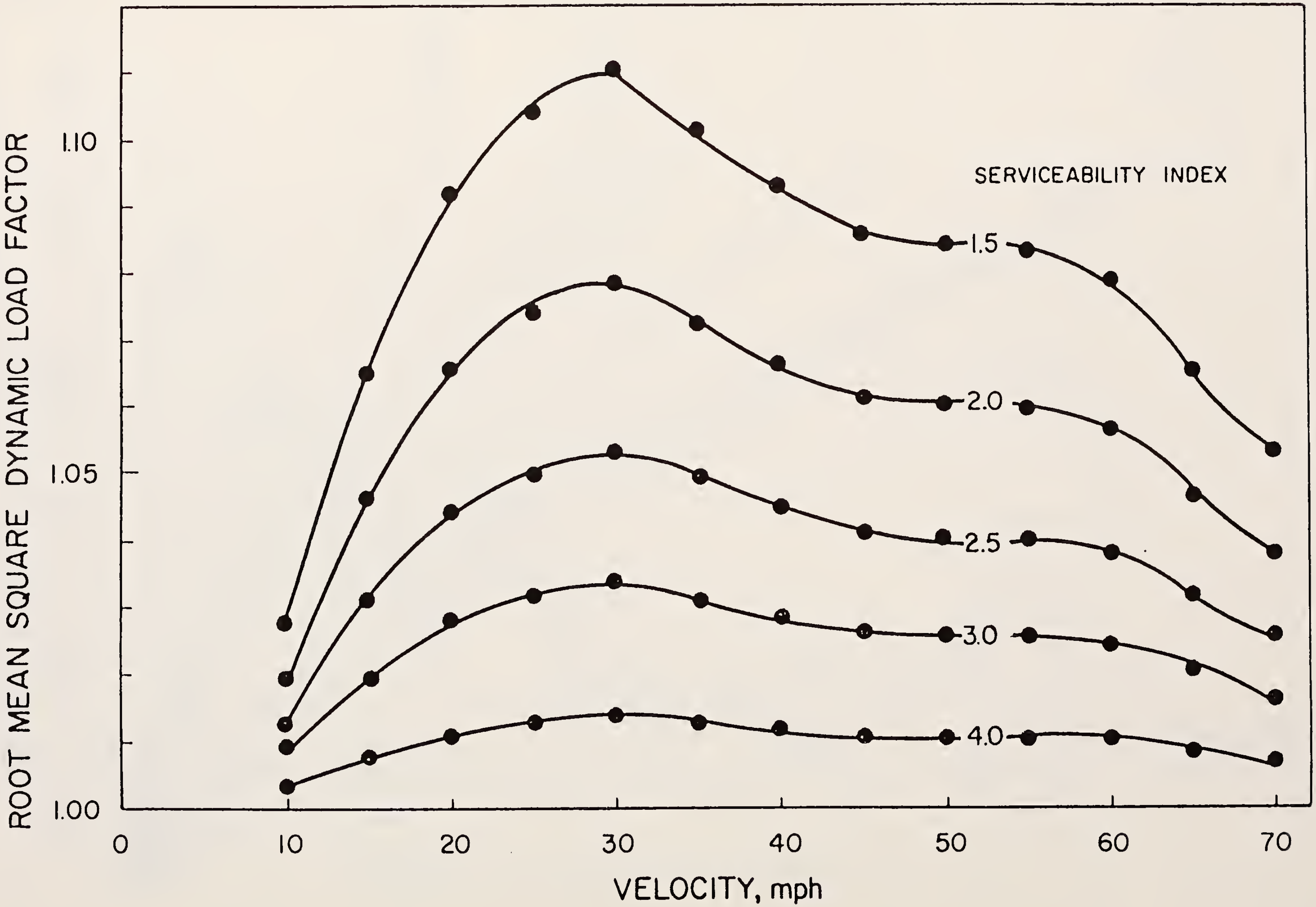


Figure 4.24. Root Mean Square Dynamic Load Factor-Speed-Serviceability Index Relationships (28).

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Chapter 5

PAVEMENT MAINTENANCE

5.1 INTRODUCTION

Several maintenance activities may have a significant influence on the deterioration of pavements subjected to excess free moisture:

1. Sealing of transverse and longitudinal joints,
2. Sealing of cracks and other surface distress,
3. Surface seal of asphalt pavements,
4. Undersealing of PCC pavements, and
5. Cleaning of subdrainage pipes.

The neglect of these maintenance activities may lead to increased moisture accelerated distress (or MAD), while their proper and timely application may prevent or retard MAD.

This chapter briefly summarizes available information concerning infiltration of surface water, sealing of joints and cracks, surface seal of asphalt pavements, undersealing of PCC pavements, cleaning of subdrainage pipes and outlets, and current maintenance practice. The information contained in this chapter was obtained from field surveys, published reports and interviews with state highway engineers. This data is valuable in the determination of feasible rehabilitation and maintenance alternatives for pavements identified as having significant moisture damage potential.

5.2 INFILTRATION OF SURFACE WATER

Surface water from precipitation infiltrates into the pavement structure through transverse joints, longitudinal joints, cracks, and permeable surface materials. Examples of water infiltration are shown in Figures 5.1-5.3. Laboratory tests have confirmed that an open joint or

crack has the potential to admit large quantities of surface water. Results given in Table 5.1 show that crack width ranging from 0.035-0.125 ins. (0.9-3.2 mm) resulted in 70 to 97 percent of the runoff entering the cracks for PCC pavements with slopes varying from 1.25 to 3.75 percent. These tests were based on slabs with no obstructions at the bottom of the cracks and a constant precipitation rate of 2 in/hr (51 mm/hr). Open cracks and joints in actual pavements, however, would generally have lower infiltration rates after available void space becomes filled with water. The rate of entry would then depend on how quickly the water can flow through or into the pavement substructure.

An infiltration study was conducted by Barksdale and Hicks (5) to obtain an indication of the percent infiltration of surface water that actually exists in the field. Tests were conducted in Georgia at two locations on an Interstate highway which has plain jointed PCC traffic lanes and asphalt concrete shoulders. Details of design, joints and infiltration data are given in Table 5.2. Prior to testing, the pavement had experienced a period of greater than average precipitation. Hence the subgrade would be expected to be in a rather wet condition. This fact may have tended to reduce the rate of infiltration since the capacity of the subgrade to accept water would be reduced. Overall 1 and 64 percent of total precipitation infiltrated into the pavement at the two sites during two rainstorms. Of course, infiltration rates greater or less than this could be expected depending on specific pavement conditions and structure.

A laboratory study was conducted (30) on the water percolation through cracks in continuously reinforced concrete pavements. An attempt was made to establish a maximum allowable crack width to limit the infiltration of water and the subsequent corrosion of the reinforcing steel. The percolation

tests of transverse cracks in laboratory constructed slabs were conducted under the simulation of ponding conditions. Salt water 3/4 inch deep was ponded over the crack and the permeability found by measuring head loss. Figure 5.4 illustrates the effects of crack widths on the time it takes the water to percolate to various depths. It can be expected that at crack widths greater than 0.01 in. (0.25 mm) the surface water will easily flow through the slab and reach the subbase rapdily. The pronounced effects of crack widths on permeability can be seen in Figure 5.5. The permeability is low at 0.01 in. (0.25 mm) width and increases rapidly at a width slightly larger than 0.02 in. (0.51 mm). This indicates that the wider cracks allow entry of significant quantities of surface water. It must be realized that these tests were run under severe ponding conditions which may rarely occur in the field. However, it was concluded that crack widths must be kept below 0.02 in. (0.51 mm) in order to prevent excessive infiltration of water, and the subsequent erosion from pumping and other problems. It was considered desirable to keep the widths of cracks to a limit of 0.01 in. (0.25 mm). The crack width required to prevent steel corrosion was so small that it was believed to be impractical to prevent water from reaching the steel. Field observations in Illinois shows that wider cracks have greater amounts of corrosion of the longitudinal steel reinforcing bars (31).

Asphalt concrete surfaces have been shown (9, 29) to have sufficient permeability to allow the entry of some water. Table 5.3 summarizes the varying degrees of permeability of water through asphalt concrete pavements with various cracks and material composition as measured by laboratory and field tests. If the surface is significantly cracked and the cracks are not sealed, surface water can infiltrate very easily.

The longitudinal joint between the traffic lane and shoulder is a most likely location for surface water to enter in large quantities if the joint is open. Considerable water infiltration into the longitudinal joint during rainstorms has been observed on many pavements during field surveys by the authors as illustrated in Figure 5.3.

In summary, available information indicates that the quantities of precipitation infiltrating into pavements are potentially high, perhaps ranging up to 60-90 percent for open cracks or joints during the initial part of the precipitation. After the voids have been filled, the amount of infiltration would depend on such factors as permeability of pavement and subgrade components, and geometry of pavement layers. Experimental data are not available as to the long term effect of joint and crack sealing on water infiltration. This type of information is definitely needed.

5.3 SEALING OF JOINTS AND CRACKS

Adequate sealing of joints and cracks is believed to be beneficial in prolonging the life of a pavement by many engineers. This conclusion is supported by general field observations and opinions but only limited comparative data is available. Results from studies in California and Illinois are cited as follows:

> "The 13-year old Manteca (CA) and 17-year old Turlock (CA) projects were almost identical in environment, construction materials, traffic and design with the exception that at Turlock, the joints were sealed during construction and apparently kept sealed since, while the joints at Manteca were never sealed. Most of the joints at Manteca (13 years) were faulted more than 0.20-inch, while at Turlock (17 years), most were faulted less than 0.15-inch and riding quality remained good. Based on the evidence from these two projects alone, it would appear that <u>joint sealing is an effective tool for retarding faulting</u>. Unfortunately, similar projects for comparison and corrobation could not be found." (Ref. 32).

The second example from Illinois is summarized in Table 5.4. The influence of sealing the longitudinal traffic lane/shoulder joint on the amount of longitudinal and area exacting showed the following:

1. Shoulder sections having granular, cement-aggregate, and pozzolan-aggregate bases with sealed longitudinal joints had much less cracking. The cracking problems of the shoulders having cement or pozzolon aggregate bases was primarily due to the deterioration of these materials,
2. Cracking did not occur in the PCC shoulder for either sealed or non-sealed sections,
3. Thick bituminous aggregate shoulders had about half the area cracking for sealed joint than the non-sealed joint.

Limited field tests showed that subgrade moisture content beneath the sealed longitudinal joint was approximately 0-4 percent lower than under the non-sealed joint. Thus some comparative data exists to support the statement that in certain instances joint and crack sealing has a beneficial effect on reducing pavement distress. Many times when joints and cracks are thought to be "sealed," they are actually not water tight seals. Thus, water infiltrates and pavement distress may occur even though the joint was thought to be "sealed." However, the existence of sealant material will at least inhibit infiltration to some extent which is beneficial.

Materials currently available for use in sealing cracks and joints can be classified as either liquid or preformed. Liquid sealants can be of the hot or cold applied type. The hot-poured sealants consist of tars, asphalts, rubberized asphalts and most recently the elastomeric polyvinyl chloride material. The hot poured materials are heated prior to placing in the joint and solidify in place upon cooling. The cold-poured sealants

such as solvent-type asphalt, emulsions and elastomers require no heating prior to placement. These materials solidify in place by evaporation of solvent, or chemical reaction. The most widely used preformed joint sealant is the neoprene compression seal. However, other types such as foams and presealed joints are available.

Many agencies attempt to seal cracks and joints to minimize or prevent both infiltration of water and incompressibles. Sealing of joints is commonly performed at the time of construction and at later times along with any cracks that develop as part of pavement maintenance practice. Resealing practice varies from annually to many years, if ever. The types of joint and crack sealants currently in use have widely varying degrees of performance. Many factors can contribute to inadequate performance of joint and crack sealants such as: sealant reservoir shape, opening and closing of joint or crack, sealant material properties and durability over time, poor construction procedures, environmental factors, and conditions and type of material which is being sealed.

5.3.1 SEALANT DESIGN

The design of the sealant must be based on the performance requirements imposed by field conditions. Many research studies and field observations have shown that the seal must be able to (1) accommodate cyclical movement caused by moisture and temperature variations of the specific pavement; (2) seal against infiltration of water (which many times contains corrosive chlorides) and incompressibles; (3) remain basically unaffected by environmental factors such as sunlight, heat, temperatures, ozone, etc.; (4) withstand the forces exerted by traffic, weather conditions, maintenance materials (cinders, sand, etc.);

(5) maintain a good bond with pavement surface sides, and (6) be economical in terms of initial cost and effective service life.

The design of a concrete pavement joint seal should consider the cyclical horizontal movement caused by both temperature and moisture variations within the concrete. The primary cause of slab movement is temperature fluctuations but concrete moisture and subgrade friction influences this movement. Seasonal temperature differences in PCC pavements vary considerably over the United States (Ref. 11). Figure 5.6 illustrates the constantly changing temperature differential at various pavement depths over a 24 hour time interval and the change in joint width with the narrowest width occurring at midafternoon as expected due to slab expansion. The Utah study (10) also shows a correlation between joint movement and slab temperature over a year time period. A New York study (14) of 12 projects were the joint movement was monitored indicates considerable variation in movement between different pavements of the same design and joint spacing (60 ft , 10 in.). A mean annual movement of 0.120 in. with a coefficient of variation of 44 percent between projects was obtained. Variation in joint opening from joint to joint in a given project is also high with a coefficient of variation of 40 percent computed on one project (Ref. 10). The variations in joint opening are believed to be the result of variations in initial cracking of the joint, subbase friction, and the coefficient of expansion of concrete.

Previous studies (6, 11) have shown that the theoretical change in slab length is a function of the overall slab length, thermal coefficient of expansion of the concrete, and change in temperature. Research indicates the computed movement as a function of slab length assuming no friction

was about 125 percent of the actual movement. The differences are believed to depend on several factors but mainly the frictional restraint of the subbase.

The mean transverse joint opening over a year or day time interval can be computed approximately using the following expression (4):

$$\Delta L = CL\,[\alpha \Delta T + \varepsilon] \qquad (5.1)$$

where

ΔL = joint opening caused by temperature change T and drying shrinkage of PCC

α = thermal coefficient of contraction of PCC (/°F) (generally 5-6 x 10^{-6}/°F)(9-10.8 x 10^{-6}/°C)

ε = drying shrinkage coefficient of PCC slab (approximately 0.50 to 2.50 x 10^{-4} in/in.)(cm/cm)

L = joint spacing (ins.)

ΔT = temperature range (for design use temperature at placement minus lowest mean minimum monthly temperature)

C = adjustment factor due to subbase/slab frictional restraint (0.65 for stabilized subbase, 0.80 for granular subbase.)

The recommended adjustement factor, C, and the drying shrinkage coefficient suggested were computed from limited field data from Utah (Ref. 10), Florida (Ref. 13), Michigan (Ref. 33), and California (Ref. 34). Using typical values for slab and temperature drop, mean joint openings are computed in Table 5.5. Values of joint openings during ten years of performance in seven states with joint spacing range within the values shown in Table 5.5 (Ref. 33). An example of the design of joint widths in a transverse joint is given in Reference 4.

The effective sealing of the traffic/lane shoulder longitudinal joint is the most difficult of all joints. An NCHRP study (5) of pavement

longitudinal shoulder joints indicates four types of movement occur: transverse movement (perpendicular to pavement edge), longitudinal movement (parallel to pavement edge), vertical movement, and rotational movement of the pavement and shoulder with respect to each other. Equations were developed to predict movement of these joints in the transverse and longitudinal directions but it was concluded that the vertical movement must be obtained from measurement. A longitudinal sealant reservoir shape of 1 x 1 in. was recommended to be capable of withstanding the severe movements at this joint.

While joints can be designed for proper width, cracks are normally very narrow and sealant is merely poured into the crack. It is not surprising that the sealant in these cracks fail rapidly. During field surveys some cracks have been observed to have been "routed" out to a width of at least 0.25 in. and then sealed. The performance of the sealant is definitely better than the normal crack filling method. Research in Minnesota concluded that a routed crack shape of 1/2 in. wide by 1 in. deep (along with surface brushing and blowing out with air prior to sealing) provided the best performance (Ref. 23).

The shape factor of the sealant has an appreciable influence of subsequent performance. In the case of poured-in-place liquid sealants, the material is stretched as the joint or crack opens causing the cross-sectional area to decrease. This opening and closing develops stresses and strains within the sealant and at the interface between the sealant and the joint or crack faces. The shape of the stretched sealant is related to sealant reservoir width and depth. In a study by Tons (7) the hour-poured sealant top and bottom surfaces become parabolic in shape as the sealant is extended. This curve-in occurs on both the top and

bottom of the sealant. Recent studies (7, 5, 8) conclude that the stress in the sealant is minimized if the parabolic shape is allowed to occur freely. This is obtained in the field by the use of a bond breaker under the sealant. The wider the seal reservoir at its minimum width, the less it will be strained on extension. Likewise Tons (7) reported that decreasing the depth also decreases the strains in the sealer for the same expansion. Therefore it is believed that the sealant reservoir width should be at least equal to the depth to minimize stress concentration. The general maximum extension of liquid sealants is in the range of 20 percent to perform adequately.

The shape of the preformed seals varies considerably. These seals are required to deform within themselves and maximize compressive forces. The preformed compression seal's performance is closely related to its ability to maintain sufficient contact with the joint walls to prevent foreign material intrusion and water infiltration. The preformed sealants are designed so that the seal will always be in compression (a minimum of 20 percent compressed from normal uncompressed width). The maximum compression of the seal is 50 percent before a rubber on rubber situation is reached. Thus the seal working range is 20 to 50 percent (3, 16, 20). A preformed sealant design example is given in Reference 4.

5.3.2 PERFORMANCE OF SEALANTS

Performance of a sealant can be characterized as the ability of the sealant to successfully function as intended over a period of time. This implies that sealant failures which prevent the material from functioning as intended are a direct indicator of sealant performance. There are basically five types of sealant failures in the liquid hot or cold poured materials as shown in Figure 5.7.

1. Adhesion Failure - This failure results when the tensile stress in

the sealer is greater than the bonding force of the sealant at the pavement joint or crack face. This produces a tearing of the sealant at the interface allowing undesirable infiltration on joint opening.

2. Cohesion Failure - This case is just the reverse of case 1. Here the bonding force at the joint or crack face is greater than the tensile strength resulting in tearing of the sealant itself. "Age" cracking is a form of cohesion failure when environmental conditions cause the surface of the sealant to become brittle and hence have a lower tensile strength.
3. Spalling-Failure - This is not actually a direct sealant failure. however, on subsequent joint opening could cause the sealant to peel away from the joint or crack interface producing an adhesive type failure.
4. Extrusion Failure - The volume of the joint or crack sealant basically remains constant as the joint or crack opens and closes. If the sealer is compressed to the degree where it protrudes significantly above the pavement surface, an extrusion failure can result. Upon protruding above the pavement surface the sealer is subject to traffic and can be spread over the surface and hence never return to the joint or crack.
5. Intrusion Failure - This condition occurs when incompressibles or debris collect on top of the sealant during a period of joint or crack opening causing a wide joint or crack. As the joint or crack closes the incompressibles become entrapped in the sealant. Stress concentrations develop on the next cycle of opening which often results in failure. This intrusion can also contribute to

extrusion because the debris displaces a certain volume of sealant which will protrude above the surface of the pavement when the joint or crack closes.

The preformed compression slabs generally fail in three ways:

1. Compression Set - This failure occurs when the seal is compressed by the expansion of the pavement slab and remains in this condition for a period of time, due to stress relaxation, the seal fails to expand as the joint opens thus failing to seal the joint as desired.
2. Web Adhesion (Sticking) - This failure is characterized by the adherence of the internal webs or sections of the seal to one another (Figure 5.8). When the seal is compressed the webs are in contact and as the joint opens the sticking forces are greater than the forces tending to expand the seal preventing the seal from exerting pressure on the slab faces. This condition is a function of material characteristics and temperature.
3. Displacement - This failure results when the seal fails to remain in its proper position in the joint. This can be caused by a number of factors such as traffic and its forcing of incompressibles against the seal, shearing action caused by slab faulting, or most commonly because of poor installation practices.

The period of acceptable performance of the various seals available and currently in use varies from a few months to several years (1, 2, 4, 3, 5, 12, 14, 15, 17, 18, 21, 22, 24, 25, 26, 27, 28). It must be recognized that the sealant performance is not only a function of material properties, but also is related to joint design, climate and construction procedures.

Asphalt cements and catalytically blown asphalts were first used for

joint sealing and they are still in use due to their relative low price. The effective life of these sealant materials is short (17, 18, 24) ranging from a few weeks after placement to approximately two years. Generally failure occurs during the first period of cold weather when the material stiffens and the joint opens. The asphalt cements are very temperature susceptible, becoming soft during warm weather and hard and brittle during cold weather. Adhesion failure is common and pieces of the sealant break off leaving the joint or crack open during cold weather. During warm weather when the materils is soft, embedment of incompressibles easily occurs. This embedment also promotes extrusion failures. Cohesion failure is also common to this type of sealer. The blown asphalts are tougher but still exhibit failure in a similar way (18).

Another hot-pour commonly used for joint sealing is rubberized asphalt. An improved rubberized asphalt is also in use that contains a higher rubber content. The original and improved versions of this type of sealant perform over periods ranging from a few months to about two years (1, 12, 14, 17, 18). The problems of adhesion, cohesion, and extrusion as mentioned previously are prevalent modes of failure in both rubberized sealants. The intrusion of incompressibles is common in both sealants but the improved version tends to reject this foreign material better (28). The temperature susceptibility of these materials is improved over the asphalt cements and blown asphalts, but the brittleness in cold weather and softness in hot weather still exists. These materials appear to have the ability to "heal" themselves in hot weather under traffic action and readhere to joint and crack walls. However, this action may only obscure the adhesion failures (25). In one study (24) of various sealant types the improved rubberized asphalt gave the second best performance, exceeded only by a compression seal. However, this

material had 69 percent failure after 2-1/2 years and thus the use of these materials would require frequent resealing maintenance if used.

One hot-pour that shows promise as an effective sealant is a polyvinyl chloride (PVC) which when heated polymerizes into a resilient sealant. This type of sealant is currently under evaluation by various agencies. The effective life of this sealant is estimated to be 3^{+} years (17, 18). One manufacturer is offering a ten year service life warranty if the PCV material is placed in the joint to recommended specifications. The material has low temperature susceptibility and good resistance to foreign material intrusion. Failures which do occur are usually ones of adhesion (17, 24). In one recent study (25) deep cracks in the PCV material existed at an age of 4-1/2 years with the surface being cracked and brittle. Another study (24) showed an 85 percent failure rate of a PVC sealant at the end of a 2-1/2 year interval. These variations in performance indicate that additional information and evaluation is needed.

Two-component cold-poured elastomers such polysulfides, urethane, polyurethanes, and chloroprene have been used. The success of these sealants is limited with commonly reported failures within 2 years (1, 2, 14, 15, 17, 25, 26, 28). The polysuflides commonly fail in adhesion (15, 25, 28) but cohesion and intrusion also occur. The surface of this material seemed to crack with age and progress until completely through the full depth (1, 14). The urethanes like the polysulfides exhibit adhesion failure. There is also the problem of traffic pulling this sealant from the joint (28) after bond failure leaving an open joint. The polyurethanes also experience adhesion failures which occur more frequently than cohesion failures (25). This sealant is quite resilient and it is believed that better performance could be obtained if the bond

failures could be reduced by better joint width design. The chloroprene elastomer evidenced many bond and cohesive failures in a recent study (26) within the first year and 90 percent failure at 54 months. This material deformed and shriveled with age and effective life was determined to be 12 to 20 months.

The open cell preformed neoprene seal has been one of the best performing of available seals. It has a service life of 3 years to 10^+ years (1, 4, 14, 15, 28). Of the modes of failure previously discussed, loss of bond to the slab due to spalling and compression set are common causes of preformed open-cell sealant failure (17, 14). Other causes of failure seem to be construction related, such as improper placement (too high or low) in joint, stretching, or twisting of the sealant. A recent study (24) rated open-cell neoprene seals as the best of practically all currently available sealants after 2-1/2 years of performance. Another recently available seal is the closed cell compression sealant. A Georgia study (25) indicates that this seal gave good performance and was capable of providing a watertight joint whereas the open cell did not. The open cell seals tend to fail as the joint faults, but the closed cell compression sealant can handle significant faulting without failure. The preformed seals have low temperature susceptibility and resist foreign material intrusion, but sometimes fail at the seal-slab interface allowing water and fine sand to accumulate there. Generally acceptable performance can be expected if proper joint shape and movement design and good construction practice are followed.

5.4 SURFACE SEAL OF ASPHALT PAVEMENTS

The infiltration of water through fine cracks or voids in asphalt pavement surfaces may lead to several types of distress and complete

breakup of the surface course. There are several types of surface seal coats that have been previously used to prevent water infiltration (and also raveling and weathering of the surface) including the following major types:

1. Single surface treatment: An application of asphalt followed immediately by a single layer of aggregate of approximately uniform size. This seal provides a reasonable water proofing layer for 1-2 years.
2. Multiple surface treatment: Two or more single surface treatments placed on top of each other. The aggregate maximum size of each successive layer is usually one-half the previous one and the total thickness is about the same as the nominal maximum aggregate size of the first surface treatment. The multiple surface treatment provides a better and longer lasting water proofing course than a single surface treatment (Ref. 35).
3. Emulsion slurry seal: A mixture of slow setting asphalt emulsion, well-graded aggregate, water, and mineral filler. The slurry is a fluidlike mixture that can be spread in thin layers over an existing surface. The slurry fills cracks and provides a hard, dense surface that prevents moisture and air infiltration into the pavement (Refs. 35, 36).
4. Fog Seal: A light application of slow-setting asphalt emulsion diluted with water. It seals small cracks and surface voids providing a somewhat water-proof layer over a 1-2 year period.
5. Thin asphalt concrete "blanket": A thin 3/4-1 in. asphalt concrete layer is used by some agencies to seal the surface and retard deterioration.

6. Plant mix seal coat (or Porous Friction Surface Course): An opengraded, free-draining asphalt mixture that is placed on pavement surfaces in thin layers (3/4-1 in.). The surface is relatively coarse and open texture but the bottom of the layer is a voidless mixture of asphalt and aggregates which prevents water infiltration downward through the seal and the water runs off laterally (Refs. 37, 38).

The use of a rubber-asphalt binder for seal coat construction is described in a recent Implementation Package (Ref. 38) from the Federal Highway Administration. The amount of granulated rubber (reclaimed from discarded automobile tires) is approximately 25 to 30 percent of asphalt content. It has been used successfully in seal coat construction in Arizona and has resisted reflective cracking successfully.

A number of agencies use one or more of these seals to provide a waterproof layer and wearing course on asphalt traffic lanes and shoulders. California, for example, places a fog seal on asphalt shoulders every 3-5 years. The life expectancy of their shoulders ranges from 15 to over 20 years. A photograph of a just sprayed fog seal shoulder that is 20 years old on I-80 near San Fransisco is shown in Figure 5.9.

On the other hand, many agencies do not seal their asphalt pavements and many times this leads to excessive cracking, moisture infiltration, and eventual breakup within 10 years as shown in Figure 5.10. This shoulder is 9 years old, consists of 8 inches of asphalt stabilized aggregate, and is located in a mid-Western state. The shoulder structure is adequate, but its open porous mixture has allowed water to infiltrate and in combination with freeze-thaw and traffic loads have led to total breakup of the shoulder.

The performance of these seals has been highly variable due to design and construction problems. Fog seals in traffic lanes under moderate traffic last about 1-2 years if the existing surface is not badly deteriorated. If placed on shoulders with only occasional traffic they generally provide 2-4 years of reasonable water proofing protection. The slurry seal may last 4 years or more under moderate traffic and longer on shoulders if the existing surface is not badly cracked. Porous friction surface courses and thin asphalt concrete overlays generally provide between 3-10 years of water proofing protection under moderate traffic if the mixture is designed and constructed properly.

5.5 UNDERSEALING

Undersealing of portland cement concrete pavements to fill voids beneath the slabs has been performed for many years by some agencies. A variety of materials have been used including asphalt cement, asphalt emulsion, lime and cement slurry. For example, District 4 of the New York Department of Public Works (now the New York DOT) has conducted an extensive undersealing program since 1948 on many miles of badly pumping concrete pavements (Ref. 39). The pumping was due to water penetrating the joints and saturating the foundation soils. The pumping could be observed every time a loaded truck passed over a joint after a heavy rain. The water was forced upward through the joints and cracks under high pressure above the slab surface. This resulted in extensive slab cracking and rocking slabs. An extensive undersealing program using a rapid setting asphalt emulsion was begun using both gravity flow into joints and also pressure. Large quantities of materials were sometimes required (as much as 200 gal was pumped into the pavement at some joints). The asphalt

emulsion was found to penetrate 1-2 in. into the soil beneath the slab and prevented further pumping. However, the undersealing process had to be repeated at various times after the initial underseal to prevent further pumping. In one county the average underseal cost of over 90 miles of pavement in the early 1960's was $100,000/mile. If these pavements were allowed to break up, the cost of replacement was estimated to be many times greater.

A recent study conducted by Purdue University in cooperation with the Indiana State Highway Commission is examining the effects of undersealing of continuously reinforced pavements (along with other maintenance procedures) on a section of I-65 south of Indianapolis, Indiana (Ref. 41). This project was surveyed by the authors in April 1977 and discussions held with engineers from the Indiana State Highway Commission. Results after only 2 years indicates that undersealing has a beneficial effect on reducing the typical edge "punch out" failure on CRCP and also in decreasing the failures around existing patches. The undersealing procedures consisted of pumping hot asphalt cement under the pavement through 3 in. diameter core holes in the center of the outside lane. The asphalt forms a waterproof layer that assists in reducing the erosion of the subbase from pumping. Hot asphalt was pumped through a hole until either the pavement or the shoulder lifted approximately 1/8-1/4 in. as measured by a gauge resting on the shoulder. A 2 in. wooden pole was quickly inserted into the hole after pumping was completed. After the asphalt had cooled a 2 in. wooden plug was driven into the hole. The holes were drilled approximately 8 ft apart and about 4 ft from the lane edge. The quantities of asphalt pumped into the slab shows that there were large voids under the pavements (as much as 50 gal were sometimes pumped into a given hole).

One slab area that had been undersealed was opened up and the asphalt was found to be continuous through ranging from 1/8-3/8 in. thickness.

Specifications and procedures for undersealing are available from various agencies that presently underseal. It appears that undersealing is one maintenance procedure that should be further carefully evaluated for its potential to decrease moisture accelerated distress.

5.6 SUBSURFACE DRAINAGE CLEANING

Very little, if any, information can be found concerning the cleaning of subsurface drains in pavement systems. However, considerable work has been done in the area of agricultural subdrainage cleaning. Poor drainage performance may be the result of either clogging or sealing. A close examination of the drainage system is necessary to determine the problem. It may be necessary to excavate the drain line for visual inspection at various locations in order to fully diagnose the problem.

Grass, MacKenzie, and Willardson (42) recommend that the following observations should be made to determine if the drain is operating properly:

1. Outflow observations to determine discharge rates.
2. Water table observations to determine if the water table over the drain is lowered to drain depth shortly after rain stops.
3. Chemical observations to determine if chemical precipitates are clogging the drain.

Drainage inefficiency is normally caused by roots, silt, or chemical deposits of iron, manganese, gypsum, or calcium carbonate.

Several methods of cleaning subsurface drainage systems to restore them to full efficiency have been used in agriculture with various degrees

of success. The method used depends on the nature of the obstruction. Long flexible rods with cutting blades have been used with limited success to remove roots and silt (42). A 2 percent sulfur dioxide gas and water solution is effective for removing iron and manganese deposits from drains (42).

Grass and Willardson (43) and Ford (44) have reported on the use of pressure water jetting for cleaning subsurface drains. Grass and Willardson (43) used high-pressure water jetting to successfully remove silt, roots, and mineral deposits from clay, concrete, and plastic drain pipe. The high-pressure jetting method of cleaning drains is also used in the Netherlands, where it is referred to as "syringing" (43). Two types of nozzles, referred to as "spoutnoses," have been used in the Netherlands. One type, which had one forward directed jet and three backward directed jets for removing silt and iron deposits was operated at a low pumping pressure, 176 to 294 psi at the pump and 59 to 88 psi at the nozzle. A second type of nozzle had one forward jet, two side (90 deg) jets, and three backward directed jets.

Grass and Willardson (43) used a high pressure which was 1029 psi to 1176 psi at the pump and 441 psi at the nozzle. They were able to clean drains which contained large amounts of sand.

High-pressure equipment utilizes the macerating action of high velocity water jets to dislodge and move obstructions. Water turbulence created within the drain pipe by jets of water exiting at varying angles around the circumference of a nozzle and at points along its longitudinal axis, stirs and moves silt or mineral compound deposits. Water exiting from the backward directed jets of the nozzle creates enough force to propel the nozzle and hose up the drain.

Two nozzle types are commonly used to clean drain lines, the "cleaning nozzle" and the "penetrator nozzle." The cleaning nozzle has five jets located midway between the nose and rear of the nozzle. The jets are located around its circumference and are aimed 90 degrees to the longitudinal axis, Figure 5.11. These jets have a scouring effect on the interior surfaces of the pipe, as well as on the joints and openings. An additional five jets, located around the rear of the nozzle, at points circumferentially midway between the forward jets, are aimed 30 degrees to the longitudinal axis to give good cleaning coverage. The diameters of orifices in the rear and midway jets are: 0.1990 and 0.9995 in., respectively. The purposes of the rear jets are to propel the nozzle forward, to create high turbulence within the drain, and to provide a sufficient volume of water to suspend the dislodged material until it arrives at the access opening in the drain.

The nose of the penetrator nozzle has a single jet that is effective in disrupting silt and mineral deposits and cutting through accumulations of roots. The penetrator nozzle has five rear jets pointed at a 15 degree angle to the longitudinal axis. This low jet angle gives the nozzle greater forward thrust and a flatter cutting angle to penetrate accumulations of roots, silts, etc.

High pressure jet cleaning can be used to effectively clean pipe sections up to 700 ft long. The cost of pipe cleaning in California is between 12 and 17 cents per foot (43).

Figure 5.12 shows the change in discharge rate as a result of high-pressure water jetting. After 21 days the jetted drains discharged at a rate 45 percent greater than the nonjetted drains.

Ford (44) did similar cleaning of drainage pipe in Florida with low

pressure water jets. The system was tested on 102 drains with considerable success. Outflows rates were noted to increase considerably after water jet cleaning.

In conjunction with pressure jet cleaning Gross, MacKenzie, and Willardson (42) have found that manganese and iron deposits can be removed from drains by using a 2 percent solution of sulfur dioxide gas and water. Sulfur dioxide treatment in some drainage systems have caused flow rates to increase as much as 200 percent depending on the severity of clogging.

In general, it would appear that pressure water jetting could be adapted to cleaning subsurface drainage pipes in pavement systems.

5.7 CURRENT MAINTENANCE PRACTICE

Maintenance procedures and policy vary from state to state and even from district to district within a given state. The policy also varies depending on pavement classification (i.e., Interstate, primary, and secondary). Available funding for maintenance appears to be the major factor in how much maintenance is actually performed.

An indication of the extent of maintenance applied, that is related directly to preventing moisture accelerated damage, can be extracted from a field study conducted by Darter and Barenberg (45). Over 70 projects located in 15 states across the U. S. were surveyed. The projects ranged in age from 6 to 34 years with a mean of 15 years and included all major types of pavements. Most of the projects were Interstate or high type pavements. During the field survey each pavement was examined as to its existing condition relative to sealed joints and cracks, undersealing when pumping existed, and preventative seals when raveling and weathering existed (on asphalt pavements). Overall results are summarized in Table 5.6 for the five types of pavements.

1. Transverse Joint Sealing: Only 25 and 46 percent of jointed plain and reinforced concrete pavements had a reasonably effective joint seal. Although most of the joints had some sealant present, it was usually in poor condition and water could freely infiltrate the joint.
2. Longitudinal Lane/shoulder Joint: The percentage of projects having an effectively sealed joint ranged from 33 to 40 percent. These percentages include those projects having curbs and those having a tight (but unsealed) joint. One exception existed where 87 percent of flexible pavements had an adequately sealed joint overall. Many of these did not have a lane/shoulder joint and these were identified as "adequately sealed." Almost no project had a sealed longitudinal joint when the traffic lanes were PCC and the shoulder was asphalt concrete. These joints were usually wide and water could freely infiltrate into the pavement.
3. Undersealing: The percentage of projects that were undersealed when pumping was visually noticeable ranged from 0 to 20 for the three concrete pavement types.
4. Surface Seals: A few asphalt surfaced travel lanes or shoulders received surface seals when raveling and weathering became evident. The percentage ranged from 0 to 40.
5. Cleaning of subdrainage Outlets: Only a few of the projects contained subdrains and thus maintenance practice could not be evaluated. However, subdrains on several projects were recently examined in one state and most of the outlets were completely clogged with soil, weeds, trash, etc. When the clogging

was removed, water flowed freely and rapidly from the subdrain outlet for several minutes.

These data indicate the following about maintenance practices for high type pavements ranging in age from about 6 to 35 years:

1. Relatively few exhibit evidence that sufficient maintenance has been applied to prevent moisture accelerated distress. A large majority of these pavements have either none or poor joint and crack sealants, undersealing, surface seals, or free flowing subdrain outlets.
2. Apparently only limited moisture damage prevention type maintenance is being applied to these pavements, and/or when it is applied the materials used have such low durability resistance that they deteriorate rapidly.

These results are reflected in a state survey on pavement joint and crack sealing materials and practices by Cook and Lewis (9). The states were asked: do we seal joints to protect against water or against intrusion of incompressibles? Seventy percent felt that incompressible were the major problem. Two respondents questioned the value of sealants at all.

5.8 SUMMARY

Review of available information and current practice reveals that there are several maintenance activities available that if properly applied would significantly affect the occurrence and propagation of moisture accelerated distress. Review of current practice indicates however that the application of such activities is limited and usually inadequate.

Tests indicate that surface moisture can infiltrate freely into non-sealed joints and cracks having widths greater than approximately 0.02 in. (0.51 mm). Sealants reduce the infiltration rate initially but their

effect reduces with time as they deteriorate or lose bond with the pavement. A large majority of pavements of all types do not have effectively sealed joints and cracks, undersealing even when pumping is evident, or protective seals when raveling or weathering begins. When subdrains exist in a pavement, the outlets are usually clogged so as to prevent free flow drainage. Typical joint and crack sealants have little durability resistance and hence fail shortly after being applied and allow water to infiltrate.

The following procedures should be considered in future studies as possible maintenance procedures to minimize the occurrence of moisture accelerated distress:

1. Extensive sealing of joints and cracks with high quality sealants to prevent water infiltration.
2. Application of protective seals to asphalt surfaces to prevent infiltration through the surface and minimize raveling and weathering of the asphalt surface.
3. Undersealing of Portland cement concrete pavements to minimize pumping and loss of support.
4. Cleaning of subdrainage outlets and drains when they exist to allow free flow drainage of infiltrated water.

Further information should be collected and tests conducted during the field trips concerning the value of each of these maintenance practices to reduce moisture accelerated distress. They should be considered as possible alternatives to, or in addition to major rehabilitation of shoulders and/or placement of subdrains.

Table 5.1

Infiltration into Surface Cracks of Portland Cement Concrete Pavements* (Precipitation Intensity 2 inches/hour)(9).

Crack Width (inches)	Pavement Slope (percent)	Percentage of Runoff Entering Crack
0.035	1.25	70
0.035	2.50	76
0.035	2.75	79
0.050	2.50	89
0.050	3.75	87
0.125	2.50	97
0.125	3.75	95

*Research by University of Maryland (laboratory test data)

Table 5.2

Summary of Water Infiltration Measurements on Georgia I-85 (5).

CHARACTERISTIC	SITE A (Milepost 59.5)	SITE B (Milepost 55)
Pavement	PCC	PCC
Thickness	9 in.	10 in.
Base	3 in. Bit. Stab. Soil Aggregate	6 in. CTB
Subbase	5 in. Crushed Stone	9 in. Select Borrow
Shoulder		
Surfacing	1.5 in. AC	1.5 in. AC
Base	6 in. C.T. Aggr.	6 in. C.T. Aggr.
Subbase	1.5 in. Class I-B	2.5 in. Drainage Layer
Subgrade Soil	Micaceous Silty Sand (Fill)	Micaceous Silty Sand (Fill)
Longitudinal Grade	+ 0.79%	+ 0.96%
Pavement Cross Slope	1.18%	1.07%
Shoulder Cross Slope	4.20%	3.89%
Transverse Joint Spacing	30 ft.	30 ft.
Transverse Joint Width	1/4 in. (sealed)	1/4 in. (sealed)
Longitudinal Pavement-Shoulder Joint Width (Pavement to Shoulder)	1/4 in. (not sealed)	<1/8 in. (not sealed)
Vertical Drop at Pavement Edge	≃ 1 in.	< 1/4 in.
Mainline Pavement Faulting	≃ 1/4 in.	< 1/10 in.
Pumping Condition	Bad	None
Results		
Infiltration		
Drainage Area[1]	312 sq. ft.	306 sq. ft.
Infiltration:		
Feb. 3, 1975	+ 3% (78.8)[2]	+ 64% (55.6)
Feb. 22, 1975	- 2% (39.8)	+ 64% (18.1)

1. Drainage area was determined by fine level grid.
2. Cumulative average rainfall on drainage area during one hour measurement period (in.^{3}).

Table 5.3

Permeability of Asphalt Concrete Pavements (9).

New Pavements

Source of Information	k (in./hr)
Kari & Santucci (US 101)[x]	75
Kari & Santucci (US 101 - left wheel path)[x]	23
Kari & Santucci (US 101 - between left & right wheel path)[x]	45
Kari & Santucci (US 101 - right wheel patch)[x]	30
Cedergren	50
Reichert (Lessines, Belgium)	78
California Division of Highway Specifications	20

[x]Air Permeability

Old Pavements

Source of Information	k (in./hr)
Baxter & Sawyer (laboratory tests)	0.0001
Tomita (USNCEL, laboratory tests)	3.00
Breen (University of Connecticut - traffic lane)	0.75
Breen (University of Connecticut - shoulder)	2.25
Reichert (Lessines, Belgium)	3.50
South Africa (cracked surfaces)	1.00

Table 5.4

Effect of Sealing the Longitudinal Pavement-Shoulder Joint on Longitudinal and Area Cracking (19).

Base Material	Design (inches) (1)	Longitudinal Cracks (2) (Inner Edge) Sealed	Unsealed	Area Cracking (3) Sealed	Unsealed
Portland Cement Concrete	8-0-4 (4)	0	0	0	0
Bituminous-Aggr. Mixture	8-0-4 (4)	0	0	228	426
Gravel	3-7-0	2.6	130	-	-
Cement Aggr. Mixture	2.5-7.5-4	184	622	0	0
Pozzolonic-Aggr. Mixture	2.5-7.5-4	196	540	34	43

1. Surface, base and subbase thickness (inches) is given in that order; the crushed stone subbases are very open graded to permit drainage.

2. Lineal feet of longitudinal cracking per 1000 lineal feet of shoulder.

3. Square feet of area cracking per 1000 lineal feet of shoulder.

4. Shoulder section tapers to 6.5 in. at edge.

Table 5.5

Computed Maximum Joint Opening using Eq. 5.2 for a Temperature Drop of 60°F and Drying Shrinkage (4).

Joint Spacing-ft.	Joint Opening - ins.			
	Stabilized Subbase		Granular Subbase	
	Temp.	Temp. & Shrinkage	Temp.	Temp. & Shrinkage
15	.040	.050	.050	.060
20	.050	.070	.060	.080
30	.080	.100	.100	.120
40	.100	.130	.130	.170
50	.130	.170	.160	.210
100	.260	.340	.320	.420

$\alpha = 5.5 \times 10^{-6}/°F$

$\Delta T = 60°F$

$\varepsilon = 1.0 \times 10^{-4}$

Table 5.6

Summary of Projects Surveyed Having the Indicated Maintenance Activity (total no. projects = 70). Most Projects are Interstate or High Traffic Volume Freeways.

	Percent Projects Surveyed Having the Indicated				
Pavement Type(1)	Transverse Joint Adequately Sealed	Longitudinal Lane/Shoulder Joint Adequately Sealed	Cracks Adequately Sealed	Underseal When Pumping Exists	Protective Seal When Ravel/Weathering Exists
FLEX	--	33*	42	--	**
COMP	--	33	0	--	0
JPCP	25	33	17(0 sh.)	0	40(sh.)***
JRCP	46	38x	27xx(0 sh.)	20	40(sh.)***
CRCP	--	40x	xxx(0 sh.)	0	0(sh.)***

*Only 33 percent projects had sealed joints where a joint existed, but 87 percent projects had either sealed joints or no joint between the lane and shoulder.

**Only one project required a seal and it had been applied.

***Refers to asphalt surface shoulder only.

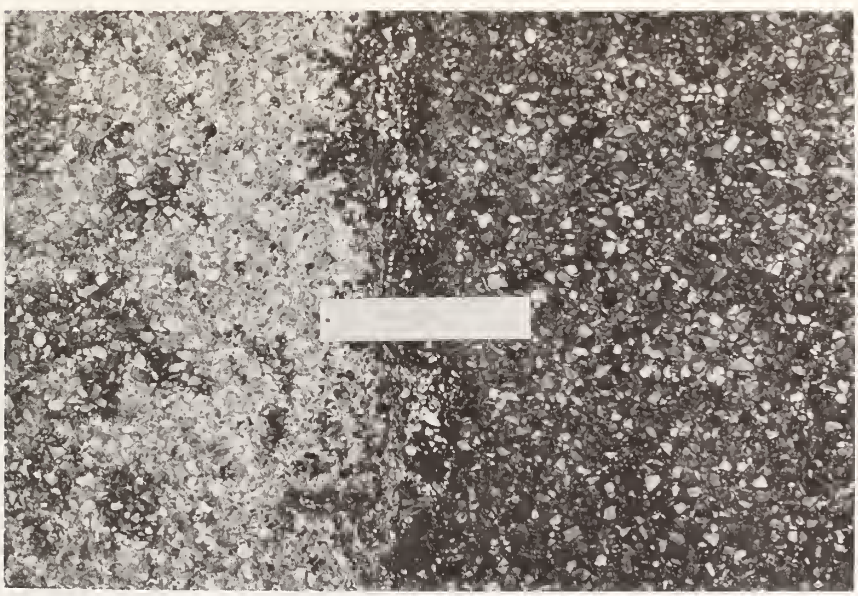
xIncludes sealed joints, curbs, and projects having very tight asphalt/PCC unsealed joint.

xxRefers only to cracks in PCC lanes. None of the projects having cracked asphalt shoulders had received any sealing.

xxxNone of the transverse cracks were sealed.

(1) FLEX: Flexible type
COMP: Asphalt concrete surface over a PCC base slab
JPCP: Jointed plain concrete
JRCP: Jointed reinforced concrete
CRCP: Contunuously reinforced concrete

(a) Overall view



(b) Closeup of typical transverse crack. Upgrade side is at right of photograph and shows water seeping into crack.

Figure 5.1. Photograph of Water Entering Transverse Cracks on Shoulder of Interstate Highway. Highway is on Approximate 3% Grade. Pavement is Dry Downgrade of Crack and Water is Still Seeping into Crack on Upgrade Side.

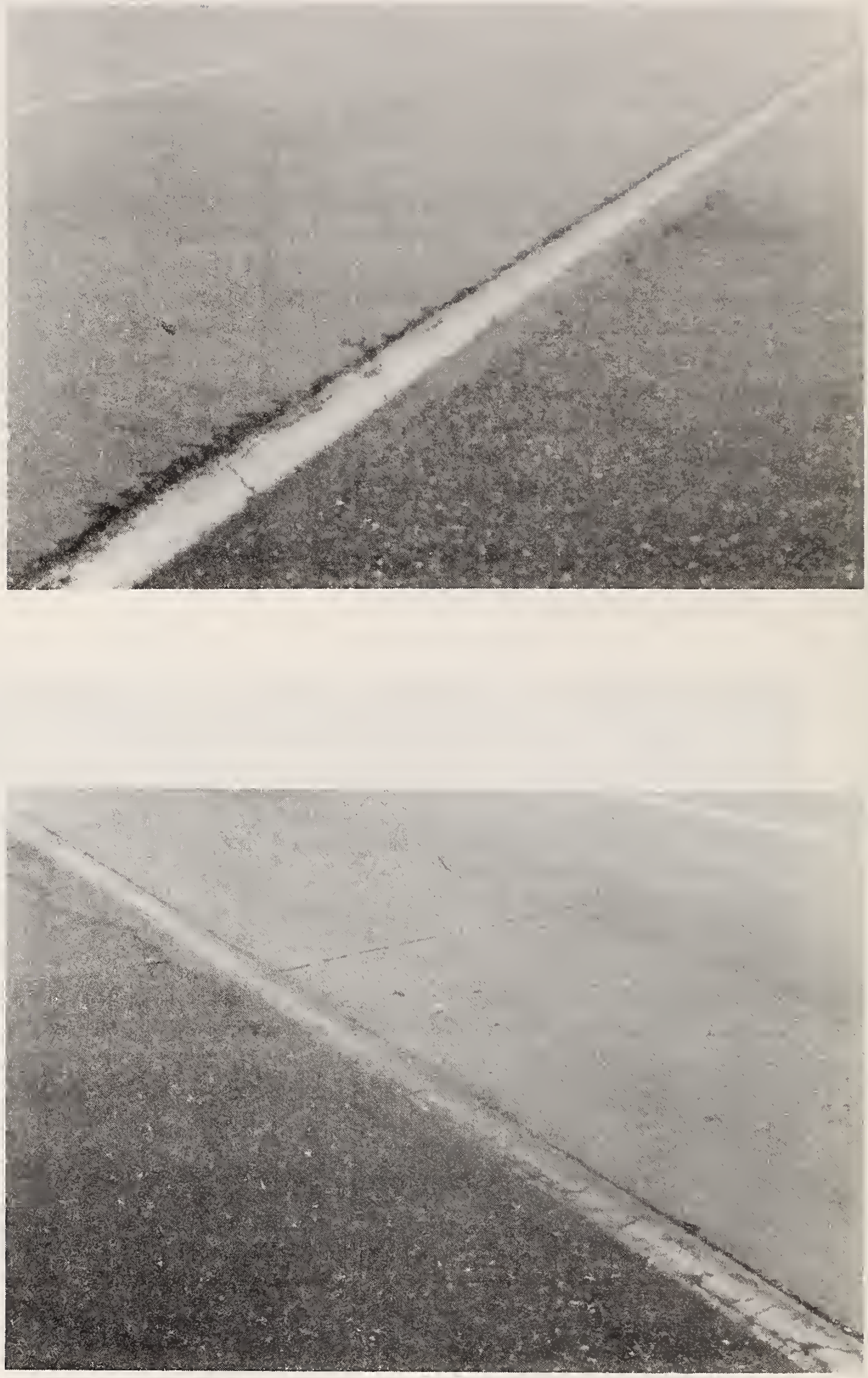


Figure 5.2. Photo Taken During Rainstorm Showing Infiltration of Water in Longitudinal Joint Between Asphalt Shoulder and PCC Traffic Lanes.

Figure 5.3. Photos Taken Several Hours after Rainstorm Showing Excess Free Moisture Infiltrated into Alligator Fatigue Cracks and Potholes which Accelerate Failure of the Pavement.

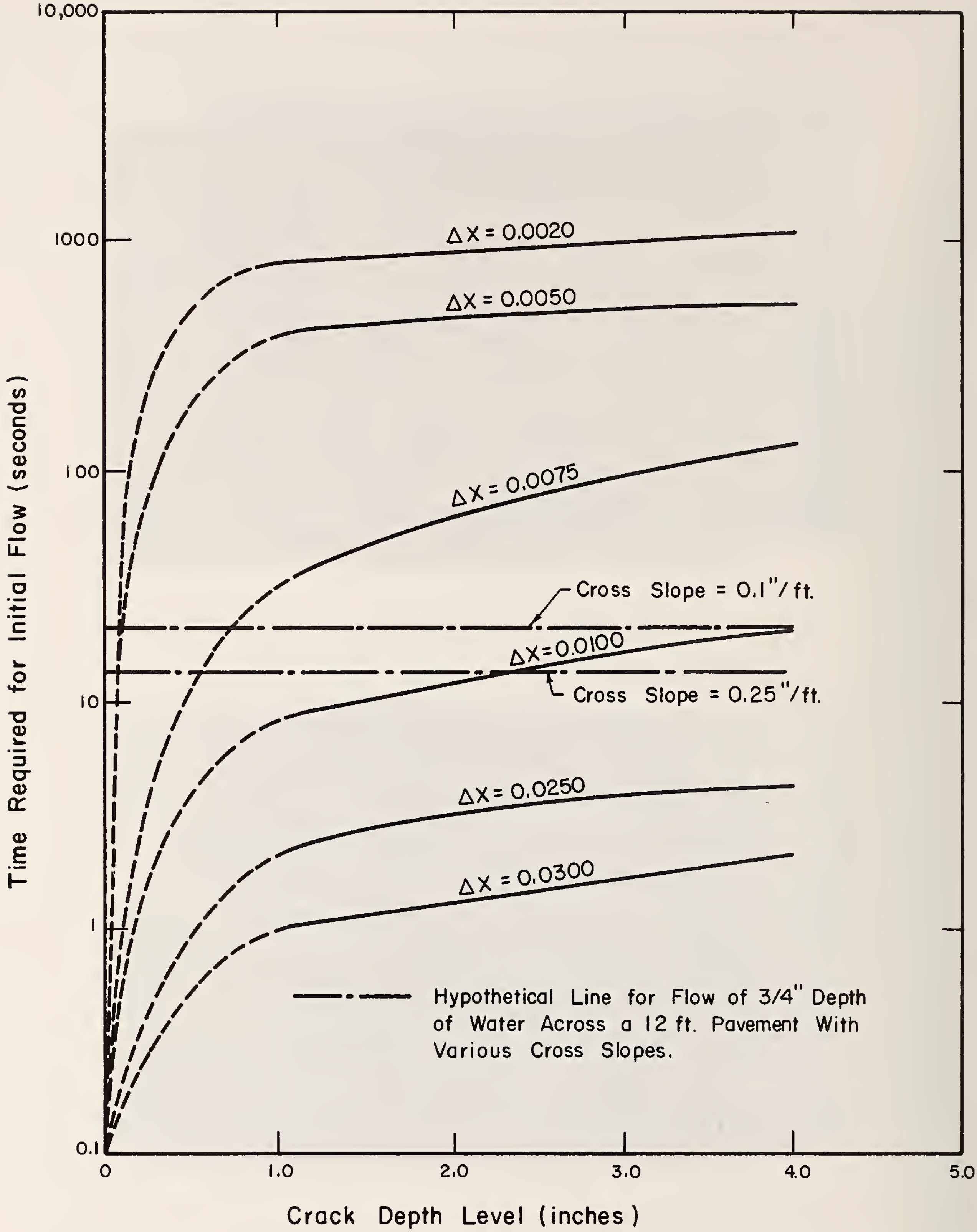


Figure 5.4. Water Percolation Rates for Various Crack Widths (30).

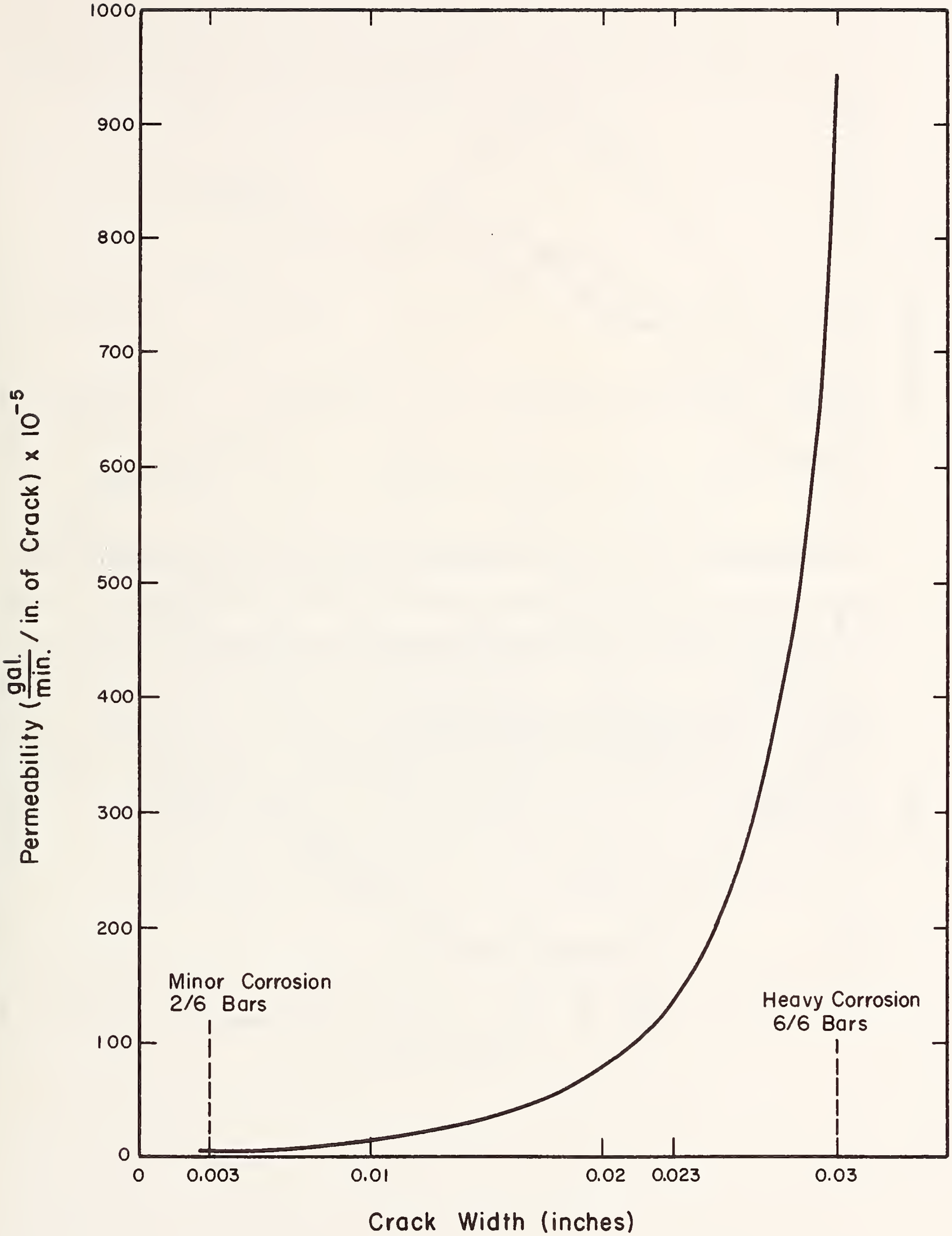


Figure 5.5. Results of Water Percolation Permeability Tests (30).

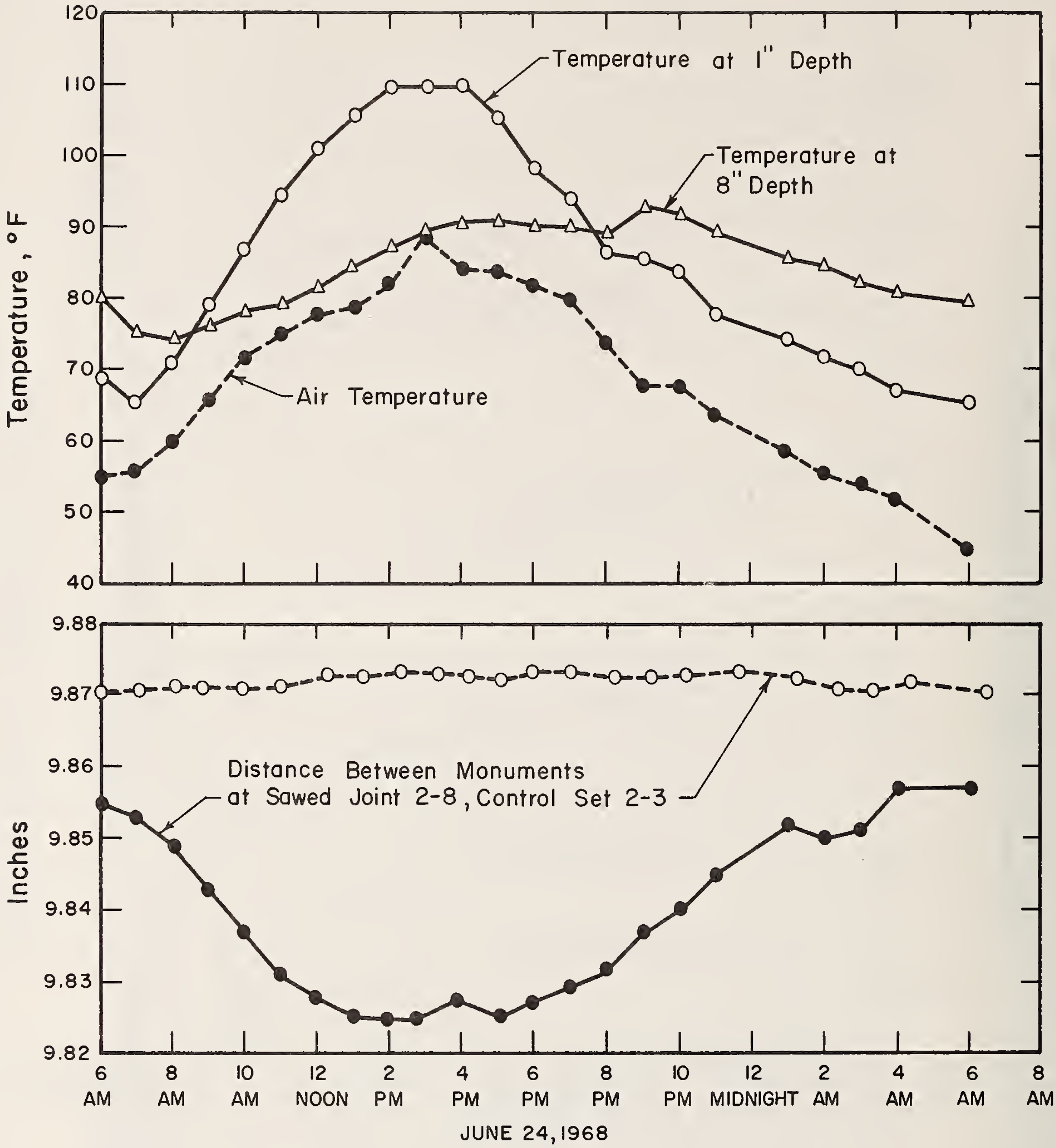


Figure 5.6. Temperatures and Movement at Typical Sawed Joint During 24 Hour Study (9 in. PCC slab with cement stabilized base and joint spacing of 12, 13, 19, 18 ft)(10).

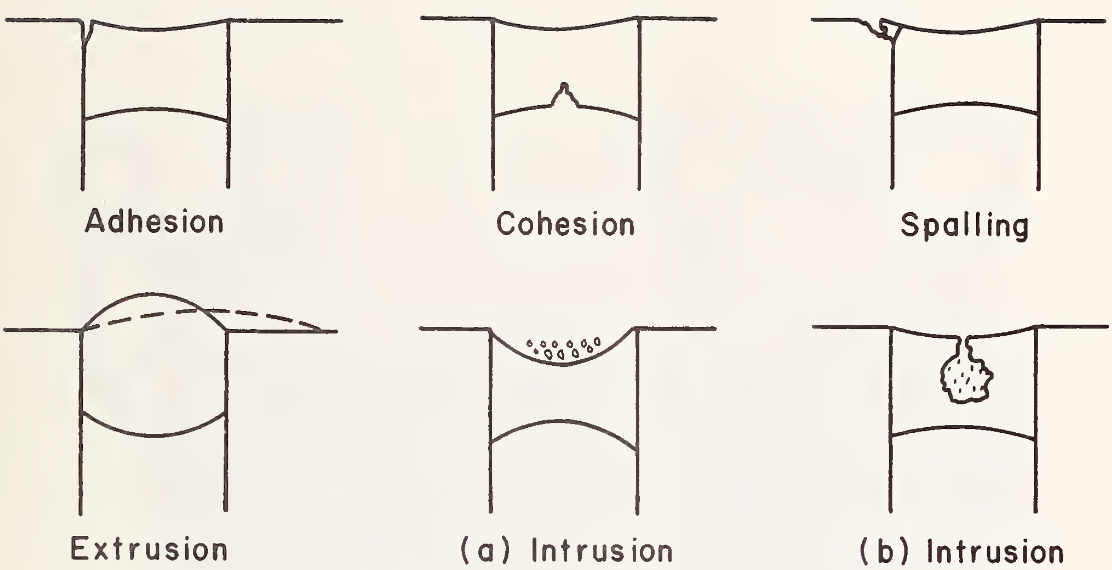


Figure 5.7. Types of Sealant Failures (2).

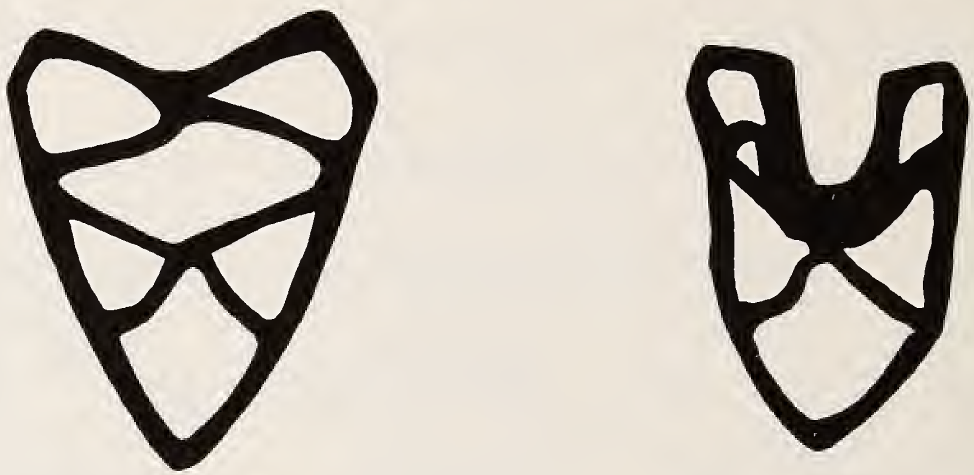


Figure 5.8. Preformed Sealer Before Installation (left), and After Service in Joint (right), Demonstrating Effect of Internal Web Sticking (14).

Figure 5.9. Fog Seal of 20 Year Old Asphalt Shoulder on I-80 Near San Francisco, CA.

a. Overall view.

b. Closeup showing separation of lifts and moisture within asphalt stabilized shoulder.

Figure 5.10. Asphalt Shoulder Located in Mid-Western State, 9 Years Old, 8 ins. No Waterproofing Seal Applied.

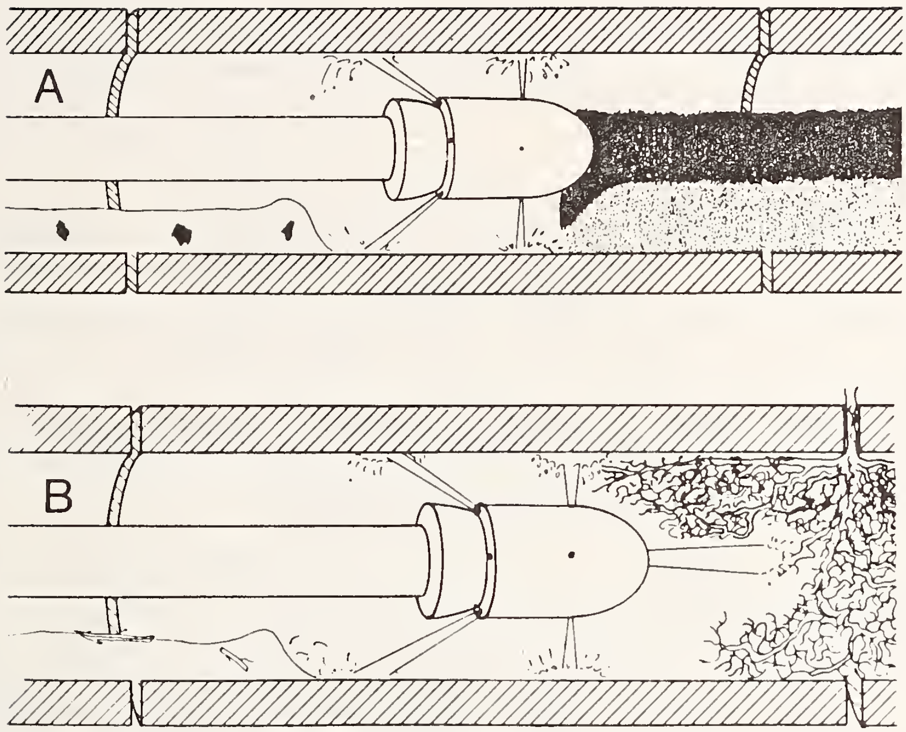


Figure 5.11. Use of High-Pressure Jet Nozzles in Subsurface Drains.
A: scouring and removing silt and mineral deposits.
B: removing roots (42).

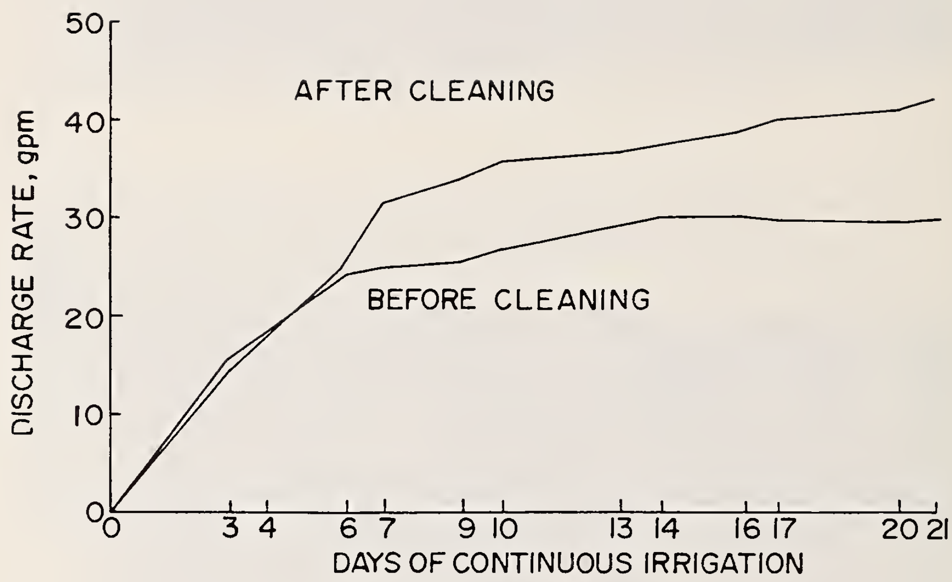


Figure 5.12. Effect of Jet Cleaning on Drain Discharge (42).

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Chapter 6

FIELD PRACTICES

6.1 INTRODUCTION

In several states considerable work is in progress to introduce new and innovative subdrainage and shoulder improvement methods in the field. With the development of new subdrainage materials and equipment substantial progress has been made to place subdrainage effectively and efficiently. It is also apparent that engineers and construction personnel are becoming more experienced and adept at placing functional subdrainage systems in the field.

In this chapter, some of the innovative subdrainage and shoulder improvement work completed by several states visited by project personnel will be shown and discussed.

6.2 SUBDRAINAGE DESIGN AND CONSTRUCTION

6.2.1 LONGITUDINAL SUBDRAINAGE SYSTEMS

One of the interesting concepts in longitudinal subdrainage is the mini-drain system used in California. Figure 6.1 and 6.2 show the collector pipe and typical outlet respectively for a mini-drain system being constructed on FAI 505 near Madison, California. The mini-drain pipe is a slotted, thick walled, plastic pipe with an inside diameter of 1-1/2 in. Figure 6.2 shows the slot size and configuration in the pipe. The longitudinal collector pipe, Figure 6.1, is placed near the edge of the pavement at a shallow depth just below the stabilized subbase. As noted in Figure 6.1 the plastic pipe is influenced by temperature changes and longitudinal expansion may be troublesome at times. However, it is

generally felt that the mini-drain functions well and can be constructed at a reasonable cost. Figure 6.3 shows the location of an intermittent longitudinal subdrain and outlet system constructed on FAI 64 east of Richmond, Virginia. The drainage systems vary from 100 ft to more than 300 ft in length and they are located mainly on long grades where water seepage from the pavement edge is a problem.

Illinois has placed considerable mileage of longitudinal subdrainage pipe. The pipe materials have consisted of short concrete sections, Figure 6.4, bituminous fiber pipe in about 12-ft lengths, Figure 6.5, and corrugated plastic tubing similar to that used in agricultural subdrainage work, Figure 6.6. Georgia has also made considerable use of corrugated plastic tubing which can be obtained in rolls of several hundred feet in length.

Although Illinois initially used subdrainage pipe with a 6-in. diameter, it has now reduced the pipe to 4 in. in diameter. Outlets are spaced at 550 ft intervals or at vertical curve sags.

Illinois and numerous other states are experimenting with various filter materials to prevent intrusion of fine material into the drainage filter envelope and subdrain pipe. The filter materials have been used to either wrap the drain pipe directly or to line the subdrain trench.

6.2.2 TRANSVERSE SUBDRAINAGE SYSTEMS

Figure 6.7 shows a transverse drainage system used on the New York Thruway between Syracuse and Albany. These granular drainage systems were placed at the transverse joints prior to overlaying the pavement and shoulders with asphalt concrete.

Figures 6.8 and 6.9 show transverse mini-drain systems constructed on FAI 5 near Red Bluff, California. These drains were placed at intervals under the flexible pavement system to intercept water flowing longitudinally in the subbase. This pavement section was showing considerable distress from pumping of fine material, rutting, and fatigue cracking.

Figure 6.10 shows transverse mini-drains installed to provide backslope stability in a deep cut in California. These drains often extend over 100 ft into the backslope. A longitudinal drain is used to collect the water at the toe of the slope. The type of drainage system shown in Figure 6.10 has been used quite successfully to decrease slope stability problems in California.

6.2.3 DRAINAGE OUTLETS

In order to insure good subdrainage performance it is necessary to provide unrestricted outlets. Figures 6.11 and 6.12 show several subdrain pipe outlet headwalls used in Illinois. Figure 6.11 shows an older formed in-place outlet headwall and splash block.
A rodent screen is specified for all outlets in Illinois.

Figure 6.13 shows an outlet for 6-in. diameter corrugated metal subdrain pipe on Route 401 east of Windsor, Ontario. The subdrainage system on Route 401 was placed at considerable depth below the pavement surface and coarse aggregate was used as envelope material.

Inspection of subdrain outlets in various states indicated that many were severely blocked with soil and debris and required maintenance.

6.3 SHOULDER DESIGN AND CONSTRUCTION

Numerous states have been experimenting with concrete shoulders tied to the mainline concrete pavement. Figure 6.14 shows a portland cement concrete pavement with a concrete shoulder on the Illinois Toll Road near DeKalb, Illinois. Corrugated rumble strips were placed in the shoulder at regular intervals. Early work in Illinois indicates that concrete shoulders perform well when properly designed and constructed.

Figure 6.15 shows a section of concrete shoulder constructed on FAI16 west of Savannah, Georgia. At the time of inspection in 1976 this shoulder was relatively new and showed no performance deficiencies. During field inspection of pavement systems throughout the United States it was observed that material integrity over the full pavement width including the shoulders was important to good performance. This observation demonstrates the need for structural compatibility as well as strength in shoulder design and construction.

6.4 JOINT SEALING

Several states have made an effort to seal transverse and longitudinal pavement joints and cracks to decrease water infiltration. Both the open cell preformed neoprene seal and closed cell compression seal have been used successfully on new pavement construction.

In the rehabilitation of existing pavement systems Georgia has found that heavy duty, self-adhesive rubberized asphalt strips, Figure 6.16, provide positive benefits. They found that the strips provide two vital properties for pavement restoration with asphalt concrete overlays as follows:

1. Waterproofing of joints and cracks.
2. Stress relief capability to prevent or reduce reflective cracks in asphalt concrete overlays.

The rubberized asphalt strips have been used to seal the shoulder-pavement-edge joint on an experimental drainage test section in Illinois (Figures 6.17, 6.18, and 6.19). The material was easily placed and bonded very well to both the asphalt concrete shoulder and concrete pavement. Two test strips about 500 ft long were placed. A liquid asphalt and sand surface treatment, Figure 6.19, was placed over the membrane in order to provide additional protection from traffic and weather. The rubberized asphalt membrane performed well during the test period and provided for significant reduction of water infiltration into the pavement system. Except for sections of the strip which have been gouged by snow plows the pavement edge joint still remains well sealed after three years.

Although the rubberized asphalt membrane is meant to be used beneath an overlay, the strips exposed to weather and traffic in Illinois are still performing quite well. Georgia has also found that traffic can operate on the rubberized asphalt strips prior to placement of the asphalt concrete overlay.

It would appear that the rubberized asphalt strips can be beneficial in waterproofing joints and cracks and reduce reflective cracking in asphalt concrete overlays.

6.5 CONSTRUCTION EQUIPMENT AND PROCEDURES

Figures 6.20 and 6.21 show some of the modern equipment which can be used to construct pavement subdrainage. This equipment can be equipped with automatic grade control systems which operate from a grade

line or laser beam. Many of the trenchers are also capable of trenching, placing subdrainage pipe or tubing, and placing granular envelope material in one operation.

Figure 6.22 shows a method of placing granular envelope material in pavement subdrainage construction and Figure 6.23 shows a small vibrating roller used for compacting the trench backfill material.

Many times the shoulder is rehabilitated during construction of the pavement subdrainage system. Figure 6.24 shows asphalt concrete shoulder material being pulverized for recycling during subdrainage installation and pavement rehabilitation in Illinois.

It is felt that equipment for rapid and economical placement of pavement subdrainage systems is readily available and efficient construction procedures for placing subdrainage systems are becoming evident.

6.6 SUMMARY

The materials, equipment, and knowledge for constructing pavement subdrainage systems in the field are available. There is also a considerable amount of field experience in the area. It would be advantageous for the various states to document and distribute their experiences in subdrainage construction.

A major finding during the field investigation was the lack of subdrainage system maintenance once it was constructed. A large proportion of the subdrain outlets were blocked with debris or soils and often it was very difficult to find them. There is a definite need to develop and define the maintenance practices required to keep pavement subdrainage systems operational once constructed.

It would appear that pavement joint and crack sealing can be helpful in decreasing the quantity of water infiltrating into the pavement system. It is especially important to seal the joint between the shoulder and mainline pavement. State agencies should be encouraged to evaluate the large number of products now available for pavement joint and crack sealing and determine if they are cost effective.

Present pavement drainage systems should be continuously evaluated by the State agencies. Through these evaluations pavement systems can be designed and built which will perform well under wet conditions.

Figure 6.1. Mini-Drain System in California.

Figure 6.2. Mini-Drain Outlet.

Figure 6.3. Intermittent Longitudinal Subdrain in Virginia.

Figure 6.4. Concrete Subdrain Pipe.

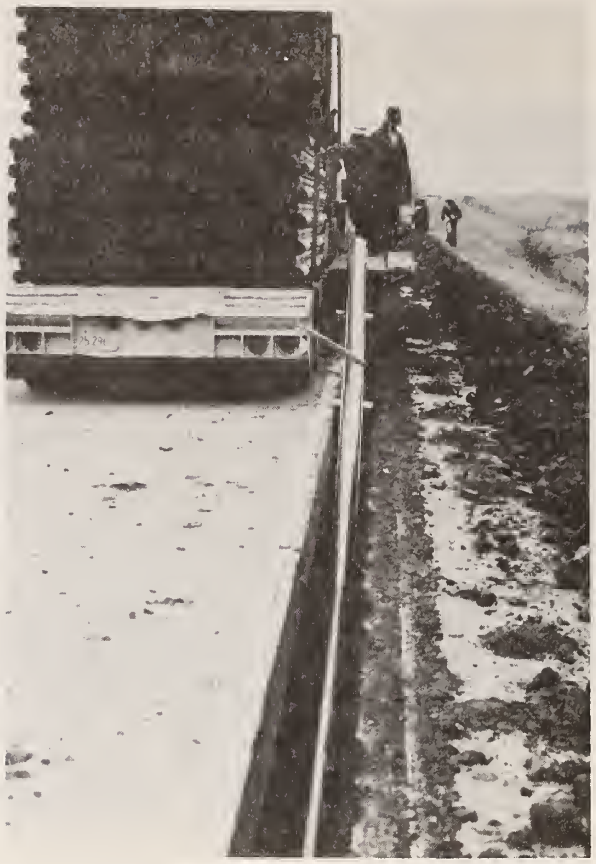

Figure 6.5. Bituminous Fiber Pipe Subdrain Installation.

Figure 6.6. Installation of Corrugated Plastic Tubing.

Figure 6.7. Transverse Drainage System in New York.

Figure 6.8. Transverse Mini-Drain System in California.

Figure 6.9. Transverse Mini-Drain Outlet.

Figure 6.10. Transverse Mini-Drain System for Slope Stability.

Figure 6.11. Formed In-Place Subdrain Outlet.

Figure 6.12. Preformed Subdrain Outlet Headwall and Splashblock used in Illinois.

Figure 6.13. Corrugated Metal Subdrain Outlet Pipe in Canada.

Figure 6.14. Concrete Shoulder on Illinois Toll Road.

Figure 6.15. Concrete Shoulder on FAI 16 in Georgia.

Figure 6.16. Rubberized Asphalt Strips for Pavement Rehabilitation in Georgia.

Figure 6.17. Placement of Rubberized Asphalt Strips.

Figure 6.18. Sealing Edge Joint with Rubberized Asphalt Strips in Illinois.

Figure 6.19. Completion of Membrane Strip in Illinois.

Figure 6.20. High Speed Trenching Machine.

Figure 6.21. Trenching Equipment for Subdrainage Installation.

Figure 6.22. Placement of SubdrainageEnvelope Material.

Figure 6.23. Compaction of Subdrainage Trench Backfill Material.

Figure 6.24. Pulverization of Asphalt Shoulder for Recycling.

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Chapter 7

SUMMARY AND RECOMMENDATIONS

7.1 SUMMARY

A review of the literature and current practices relating to subdrainage, shoulder structures, and maintenance of existing pavement systems was completed. Although the review is concerned with existing pavements, much of the material presented is also applicable to new pavement systems.

Methods of classifying various types of subdrainage are presented and the problems associated with pavement subdrainage are discussed in detail. A thorough discussion of the saturated and unsaturated flow equations for describing water movement in porous materials is presented. The concepts of heat transfer and the simultaneous movement of heat and water in soils are discussed. Procedures presently being used for subdrainage design and construction are presented.

The function, problems, and design of pavement shoulders are described in detail. Typical shoulder designs from several states are evaluated.

Maintenance practices affecting shoulder performance and moisture related distress are analyzed. A number of specific maintenance practices presently being used by state agencies to decrease the effects of moisture on pavement distress are thoroughly discussed.

7.2 RECOMMENDATIONS

Based on this literature review the following studies are recommended:

1. Determine how to best identify existing pavements which are subject to premature damage caused by poor internal drainage.
2. Determine the hydraulic and dynamic conditions which cause channeling in pavement sytems.

3. Investigate methods of shoulder treatment and suggest ways to obtain data suitable for warrants on the use of subsurface drainage.
4. Develop maintenance practices for subsurface drainage systems.
5. Develop new drainage concepts for draining water trapped in the pavement structural layers.
6. Identify pavement maintenance problems in areas with swelling soils and frost heave and develop solutions for these problems.
7. Analyze the effectiveness of curb and gutter on pavement drainage.
8. Analyze the use of various types of filter materials, drainage envelopes, and drain pipe in subdrainage design.
9. Monitor field drainage installations for operational effectiveness and contribution to improved pavement performance.
10. Determine the influence different maintenance strategies have on water infiltration and subdrainage.
11. Extend the study of traffic enroachment on pavement shoulders.
12. Develop rapid field methods for measuring the hydraulic conductivity of pavement materials insitu.
13. Develop a shoulder design procedure which considers encroachment loading, material properties, and climatic effects (hydrothermal effects).
14. Analyze the use of full width paving to include pavement lanes and shoulder as an integral structural layer in order to eliminate edge joints between the shoulder and mainline pavement.
15. Conduct workshops on the identification of pavement distresses caused by poor internal drainage, subsurface drainage design, construction, and maintenance.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

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Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

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This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

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This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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